GOVERNMENT OF INDIA
CENTRAL WATER COMMISSION

HANDBOOK FOR DESIGN OF

FLOOD PROTECTION, ANTI EROSION MEASURES & RIVER TRAINING WORKS

FLOOD MANAGEMENT ORGANISATION

NEW DELHI
DEC, 2011

Foreword

India is one of the most flood-affected countries in the world in terms of affected geographical area. There is not a single year when some or the other parts of the country gets inundated in flood water. How best to cope with floods is an age-old problem. It is a natural disaster. One way is to accept it as inevitable and learn to live with it in the best possible manner. But unlike other natural disasters such as earthquakes it is possible to manage flood to a great extent. As widely known that there are two options for flood management ie. structural & non-structural and modern flood management strategy is a judicious mixture of both of these options. This Handbook for design of flood protection, Anti erosion measures and river training works, primarily deals with structural part of flood management. The handbook deliberated on the design of various flood protection, anti erosion and river training works along with construction materials, planning and monitoring, which are integral part of any project for managing the floods and erosion. I am sure that this handbook would be of great use for the practicing Engineers at different levels in flood management works.

Preface

India has a peculiar geographical setting that there are floods in some parts and droughts in other parts and sometimes they co-exist. India has made huge investment in flood control sector since 1951 in implementing nos of flood management schemes which has undoubtedly provided lot of relief to a large population from floods. The Engineers involved in framing the project report and subsequent implementation for flood management and erosion control need to have a handbook covering the design principles, construction technique and monitoring to come out with goal oriented approach in a most reasonable and time bound manner to avoid time and cost overrun of the flood management projects. This handbook would provide better tools to plan, construct and monitor the flood management projects in a integrated manner.

Contents

List of Tables

List of Figures

Abbreviations and Common Terms

A/E	:	Anti-erosion
ASTM	:	American Society for Testing and Materials
B.C. Ratio	:	Benefit-Cost ratio
BIS	:	Bureau of Indian Standard
BS	:	British Standards
C/S Slope	:	Country side slope
CBIP		Central Board of Irrigation and Power
CC	:	Cement concrete
Cumec	:	Cubic meter/sec
cm	:	Centimeter
CWC	:	Central Water Commission
CWPC	:	Central Water and Power Commission
EN	:	European Standard
FMP	:	Flood Management Programme
G-D Curve	:	Gauge-Discharge curve
GI	:	Galvanized iron
Gsm	:	Gram/square meter
HDPE	:	High density polyethylene
HFL	:	Highest flood level
HGL	:	Hydraulic gradient line
IRC	:	Indian Road Congress
IS	:	Indian Standard
ISO	:	International Standards Organization
Km	:	Kilometer
KN:	:	Kilo-newton
LDPE	:	Low density polyethylene
LWL	:	Lowest Water Level
Mha	:	Million hectares
MHA	:	Ministry of Home Affairs
mm	:	Millimeter
MoWR:	:	Ministry of Water Resources
MPa	:	Mega Pascal
NDMA	:	National Disaster Management Authority
NSL	:	Natural surface level
PP	:	Poly propylene
PVC	:	Poly vinyl chloride
R/S	:	Raising and strengthening
R/S Slope	:	River side slope
RBA	:	Rashtriya Barh Ayog
RBL	:	River bed level
RCC	:	Reinforced cement concrete
UV	:	Ultraviolet
Zn	:	Zinc

Section-1: Introduction

1.1 General

Floods are recurrent phenomena in India from time immemorial. Almost every year floods of varying magnitude affect some parts of the country or the other. Different regions of the country have different climates and rainfall patterns and as such it is also experienced that while some parts are suffering under devastating floods, another part is suffering under drought. With the increase in population and developmental activities, there has been a tendency to occupy the floodplains, which has often resulted in serious flood damages and loss of lives over the years. Because of the varying rainfall distribution, many a times, some areas, which are not traditionally prone to floods, also experience severe inundation. Flood disasters are among the most debilitating problems faced by the country. A river bank experiencing severe erosion is shown in Figure 1-1.

Figure 1-1: River bank experiencing severe erosion

1.2 Flood damages in India

The devastating floods not only result in loss of precious human lives, cattle and damage to public and private property but create a sense of insecurity and fear in the minds of people living in the flood plains. The after effects of flood like the agony of survivors, spread of epidemic, non availability of essential commodities and medicines, loss of the dwellings make floods most feared among the natural disasters being faced by human kind.

On an average during the period 1953-2010, the floods resulted in an annual damage of more than Rs. 1800 crore besides the loss of precious human lives and cattle. The highlights of flood damages in India during the said period are given in Table 1-1.

Table 1-1: Highlights of flood damages in India during the period (1953-2010)

#	*Item*	*Unit*	*Average Annual Damage*	*Maximum Damage*	
				Extent	*Year*
1	Area affected	mha	7.208	17.50	1978
2	Population affected	million	31.019	7.045	1978
3	Human lives lost	no.	1612	11316	1977
4	Cattle lost	no.	89345	618248	1979
5	Cropped area affected	mha.	3.679	15.180	2005
6	Damage to crops	Rs crore	694	4247	2000
7	Houses damaged	no. (million)	1194637	3507542	1978
8	Damage to houses	Rs crore	276	1308	1995
9	Damage to public utilities	Rs crore	815	5605	2001
10	**Total**	**Rs crore**	**1804**	**8865**	**2000**

Note: The damage data for 2003 onwards is under validation by States. The figures are without any escalation and not at current price level. At the price level of 2010, the total flood damages have been estimated tentatively as Rs. 8,12,500 crore (approx.) considering escalation @ 10% per annum on compounded basis.

Figure 1-2: A flooded street in city

1.3 Flood prone areas in the country

Rashtriya Barh Ayog (RBA)-1980 assessed the total flood prone area in the country as 40 mha by adding the maxima of flood affected area (34 mha) in any year to the area protected (10 mha) and deducting portion (4 mha) of the protected area included in the flood affected area due to failure of protection works. The XI Plan Working Group on Water Resources compiled the area liable to floods as 45.64 mha. Subsequently, XII Plan Working Group found out that sum of maxima of flood affected area reported by States to RBA for the period from 1953 to 1978 as 33.56 mha (rounded off to 34 mha) has gone upto 49.815 mha as per data base maintained by CWC based on the flood damage data reported by the States for the period during 1953-2010.

1.4 Flood Management Programme

The Government of India is also providing financial assistance to the various State Governments through various Plan Schemes."Flood Management Programme" for providing central assistance to the extent of Rs.8000.00 crore to the State Governments was taken up during XI Plan for river management, flood control, anti-erosion, drainage development, flood proofing, restoration of damaged flood management works and anti-sea erosion works.

1.4.1 Salient features of FMP

- To avail the central assistance; the States have been advised to prepare the schemes of flood management works in an integrated manner covering entire river/tributary or a major segment. However, in case of emergent situation arising due to high floods, the works in critical reaches are taken up immediately after flood season.
- The State Governments have to ensure inclusion of the scheme in the State Plan and make requisite budget provision towards Central as well as State share on annual basis.
- While submitting a proposal, the State Governments have to ensure acquisition of land required under the scheme and submit a certificate to this effect.
- Subsequent instalments of central assistance are released on receipt of the Utilization Certificate in FORM GFR-19A submitted by the concerned Chief Engineer and the financial authority; and countersigned by the concerned Secretary of the implementing department/Finance Secretary of the state government.
- Actual expenditure incurred by the State Governments from their own resources in the financial year (in which the scheme is approved by the Empowered Committee

under FMP) would be reimbursed in the same financial year or, if the central assistance is not released in that financial year, in the next financial year, in which case requirement of budget provision may not be necessary.

1.4.2 Funds released under FMP

A total of 406 no. flood management/anti-sea erosion works of various State Governments were included under the Programme out of which 218 works have been completed upto 31-03-2011 which have provided protection to flood affected area of 18.693 lakh ha. These works have benefitted total population of 197.277 lakh in the concerned States.

1.5 Working Group on "Flood Management and Region Specific issues"

The Working Group on "Flood Management and Region Specific issues" for XII Plan was constituted by the Planning Commission in Oct, 2010. The Working Group has recommended strategies to deal with flood management during XII Plan ensuring development in the key areas in order to achieve the broad objectives, targets, associated challenges and implementation of policies by the Centre and the States. In order to have effective programme for addressing the problem of flood in the country, the following strategies are recommended to be effectively implemented during XII Plan.

- Scientific Assessment of Flood Prone Area
- Integrated Basin Management Approach
- Construction of Dams and reservoirs with adequate Flood Cushion
- Development of Detention Basins
- Drainage Improvement
- Strengthening of Organizations
- Public-Private Partnership Concept
- Inventory of Works completed by State
- Provision for adequate funds for maintenance of existing works
- Procedural Reforms
- Application of New Technologies
- Emergency Action Plans

1.6 Measures for flood management and erosion control

Different measures have been adopted to reduce the flood/erosion losses and protect the flood plains. Depending upon manner in which they work, flood protection and flood management measures may be broadly classified as under:

1.6.1 Non-structural measures

The non-structural methods to mitigate the flood damages are as under:

- Flood Plain Zoning;
- Flood Forecasting and Flood Warning;
- Flood Proofing; and
- Living with Floods.

1.6.2 Structural measures

The engineering measures for flood management/erosion control (may be classified into long term measures and short term measures) which bring relief to the flood prone areas by reducing flood flows and thereby the flood levels are:

- Creation of reservoir;
- Diversion of flood waters to other basins;
- Construction of flood embankments;
- Construction of spurs, groynes, studs etc.;
- Construction of bank revetment along with launching apron;
- RCC porcupines in the form of screens, spurs, dampeners etc.; and
- Vetivers, Newwebs, geo-bags etc.

The structural measures for flood management mentioned above are designed as per BIS codes. However, many works like RCC porcupines, Geo-textile materials, vetivers etc are not covered in the existing BIS codes.

This handbook has been designed with a view to help all the practicing engineers in the States and Central Government for design, appraisal, construction and monitoring of the flood management works covering all the relevant BIS codes, design manuals, guidelines, technical specifications for construction materials and practices etc. to meet new challenges in the flood management in India. This handbook has been provided with typical examples of civil structures to help the field professionals in standardizing the design practices and use of state-of-art technology.

Section-2: Construction Materials

2.1 General

Flood management and river training works in form of embankment, bank revetment, spurs, RCC porcupines, sluices etc. are provided to manage/control the floods, improve drainage system and to check the bank erosion. Construction of these works makes use of different kind of materials depending on the nature of problem and the structure provided.

2.2 Type of construction materials

Different construction materials have their own uniqueness and are used according to the site conditions, availability, transportability, cost effectiveness, low maintenance cost etc.

Materials like boulders, timber are in use since ages, but due to their increased usage in other sectors leading to reduced supply and environment un-friendliness, their use now-a-days is decreasing. High wear and tear of timber structures in underwater and near water situation make it less suitable for its use in anti-erosion measures.

Now–a-day's use of new innovative materials like Geo-textile in the form of Geo-textile bags, Geo-textile tubes, Sand filled Geo-mattress, Neo-web, submerged wanes, RCC porcupines is being increased in construction of revetments, spurs, groynes, embankments etc. These materials are used due to their unique characteristics like durability, resistance to chemical waste, environment friendly nature, easiness in installation etc. Different construction materials being used for structural measures of flood management are described below in detail:

2.2.1 River bed materials

Considering economy and ease in availability river bed materials including sand and boulders are widely used in flood management works. However, rounded river boulders are used in contained forms like gabions/crates but avoided for loose pitching.

2.2.1.1 Sand

The sand is used as fill material for flood embankments and spurs. The sand is also used for filling Geo-textile bags, Mattress and tubes. The sand should be coarse sand and free from organic material. Loamy and clayey type soil should be avoided.

2.2.1.2

Boulders are naturally available materials which are

Boulders

used as construction material in various works including slope protection for embankment, bank revetment, spurs etc. The boulder's shape, size, weight, gradation places an important role in its effective use. The gradation of boulders in a revetment should be well graded throughout the layer thickness. The boulders used should be angular, and regular in shape. The boulders should have sharp clean edges at the intersections of relatively flat faces. Rounded boulders should be avoided.

2.2.2 GI wire mesh

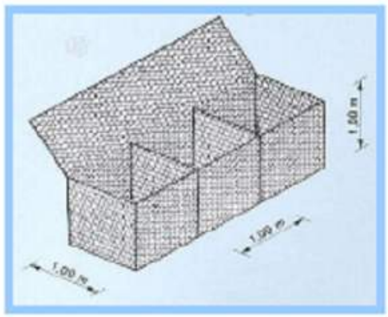


Figure 2-1: A wire-mesh gabion

When appropriate size and quality boulders are not available, gabions or crates should be used. The gabions are rectangular boxes made of hexagonal double twist steel wire mesh filled with the small size boulders/cobbles. Crates are smaller in size than gabions. Opening of the gabions or crates should be smaller than the size of smallest boulder/cobble so that they are kept intact.

2.2.3 Revet-Mattress

Revet mattress is rectangular mattress made with hexagonal double twisted steel wire mesh, where the depth is small in proportion to its length and width. Revet mattress can be differentiated with the gabions due to its lesser height. It is divided into several cells by transverse diaphragms.

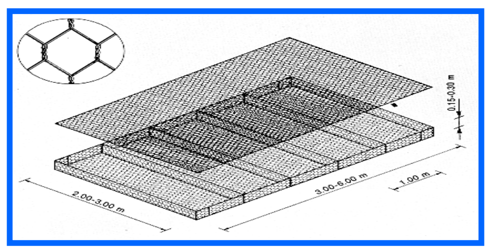


Figure 2-2: Hexagonal double twist steel wire mesh, revet mattress

2.2.4 Concrete blocks

Concrete is a composite material made from the combination of aggregate including sand, stones and a binder such as cement. Cement Concrete (CC) blocks are sometimes used in place of boulders for construction of bank revetment or slope protection of the embankment. The CC blocks may be cast in-situ and execution of works using the CC blocks is faster than the boulder works.

2.2.5 Reinforced cement concrete porcupines

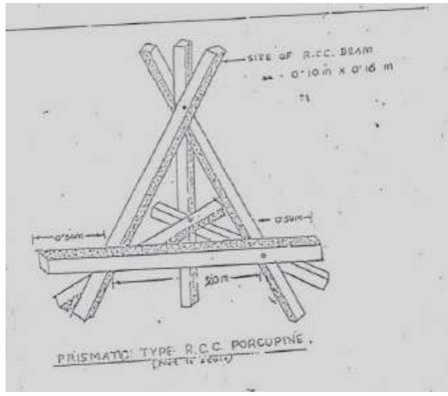


Figure 2-3: A sketch of typical RCC porcupine

Reinforced cement concrete (RCC) is mainly used for conscrcution of RCC porcupine screens due to ease of construction, cast in-situ nature, longer durability and low cost. The use of RCC is replacing the timber in construction of porcupine screens.

2.2.6 Geo-textile materials

A synthetic material in the form of strong flexible sheets either woven or non woven and permeable or water tight membranes used to improve soil quality and performance in different applications like lining, drainage, filtration, separation, reinforcement and protection have been used since long. For the specific application in flood management works, products like geo-textile bags/tubes, geo-membrane, geo-grid, geo-mattress are used. The generic name given to all these materials is referred as "GEO-SYNTHETICS". As stipulated by Indian Road Congress (IRC:SP 59) publications major important products of geosynthetic are been described in brief along with their application.

The Geo-synthetics have different application and perform different function, as described below in Table 2-1 below:

Table 2-1: Identification of the usual primary function for each type of geo-synthetic

Type of Geo-synthetic	*Separation*	*Re-inforcement*	*Filtration*	*Drainage*	*Containment*
Geo-textile	✓	✓	✓	✓	
Geo-grid		✓			
Geo-net				✓	
Geo-membrane					✓
Geo-synthetic Clay Liner					✓
Geo-foam	✓				
Geo-cells	✓	✓			
Geo-composite	✓	✓	✓	✓	✓
Geo-textile tube & bag		✓			✓

2.2.6.1 Geo-textile

The basic raw material is polymer and the most widely used are polypropylene and polyester. Based on the manufacturing process geo-textile is often categorized as woven or non woven. Woven geo-textile are manufactured by weaving weft thread through warp thread. While non woven geo-textile is produced from randomly distributed continuous or staple fibers which are bonded together chemically, thermally or mechanically.

2.2.6.2 Geo-membrane

Geo-membrane materials are relatively thin, impervious sheets of polymeric material used primarily for linings and covers of liquids- or solid-storage facilities. This includes all types of landfills, reservoirs, canals, and other containment facilities. Thus the primary function is always containment as a liquid or moisture barrier or both. Geo-membrane are of different types as per density and texture. Use of geo-membrane is rapidly increasing in areas of soil stabilization, landfills, lagoons, lining, pavement, dams and spillways etc. These membranes can be classified into HDPE (high density polyethylene) and LDPE (Low density polyethylene).

2.2.6.3 Geo-grid

A geo-grid is deformed/ non deformed grid like polymeric material formed by intersecting ribs joined at junctions. Main function of a geo-grid is reinforcement by friction mechanism. Geo-grids are:

(a) Either stretched in one or two directions for improved physical properties
(b) Made on weaving or knitting machinery by standard textile manufacturing methods
(c) By bonding rods or straps together.

2.2.6.4
Geo-net

Geo-nets are formed by a continuous extrusion of parallel sets of polymeric ribs at acute angles to one another. When the ribs are opened, relatively large apertures are formed into a netlike configuration.

2.2.6.5
Geo-synthetic Clay Liner

A geo-synthetic clay liner acts as a hydraulic barrier consists of bentonite clay or other very low permeability material, supported by geo-textile and/or geo-membrane which are held together by needling or stitching. Main area of application is in landfills, rockfill dams etc.

2.2.6.6
Geo-composite

A geo-composite consists of a combination of geo-textile, geo-grids, geo-nets and/or geo-membrane. The application areas are numerous. The major functions encompass the entire range of functions listed for geo-synthetic discussed previously: separation, reinforcement, filtration, drainage, and containment.

2.2.6.7
Geo-textile tube and bags

Geo-textile Tube is a tube made of permeable but soil-tight geo-textile (woven) and is generally filled with sand or dredged material. These tubes are generally about 1 m to 3 m in diameter, though they can be custom made to any size depending on their application. Today, geo-textile tubes ranging in diameters from 1.5 m to 5.0 m are used in many coastal and flood protection applications.

Figure 2-4: Geo-textile bag

Figure 2-5: Geo-textile tube

Geo-textile bags are made of woven or non woven geo-textile fabrics which are specially designed for good soil tightness and high seam efficiency. Geo-textile bags range in volume from 0.05 m^3 to around 5 m^3, and are pillow shaped, box shaped or mattress shaped depending on the required application. Geo-textile bags have also been used as revetment, breakwaters, etc to build structural erosion protection measures. A sample of a geo-textile tube and bag are shown in the Figure 2-4 and Figure 2-5.

2.2.6.8 Erosion control mat

Figure 2-6: Erosion control mat

Erosion control mats can be of bio-degradable or non degradable type. Erosion control mat provides immediate erosion control and high moisture content to establish vegetation. It creates hospitable conditions for plant invasion and establishment.

Biodegradable mats are made of coir or straw fibers which are used for short term erosion control unit growth of vegetation. While synthetic mats consist of UV stabilized non-degradable polypropylene fibers that are heat bonded at the contact points to provide a dimensionally stable matrix for soil erosion protection. For very high embankments and embankments with steeper slopes, synthetic mat can be reinforced with galvanized mesh with or without PVC coating. The composite nature of reinforced mat adds to the erosion control and sediment trapping function of the geo-synthetic matrix.

2.3 Technical

Materials including gabions, revet mattress, geo-textile tubes and bags are used with specific strength,

specifications and testing

durability requirements as per the proposed structure. The detailed technical specifications of these innovative materials along with the test methods and its recommended values for each parameter are being described in paras below in detail.

2.3.1 Wire mesh gabions

This work may consist of furnishing, assembling, and filling mechanically woven double twist wire mesh gabions with boulders. These specifications are mainly in accordance with International Standards EN 10223, EN 10244.

2.3.1.1 Material /structural properties

Desired properties for various components for fabrication of wire mesh gabions are as under.

2.3.1.1.1 Wire

All tests on the wire mesh, lacing wire must be performed prior to manufacturing the mesh.

Tensile strength: Both the wire used for the manufacture of gabions and the lacing, shall have a tensile strength of 350-500 N/mm^2, in accordance with EN 10223-3.

Elongation: Elongation shall not be less than 10%, in accordance with EN 10223-3.The test must be carried out on a sample at least 25 cm long.

2.3.1.1.2 Zinc coating

Minimum quantities of zinc should meet the requirements of EN 10244-2. The adhesion of the zinc coating to the wire shall be such that, when the wire is wrapped six turns around a mandrel having four times the diameter of the wire, it does not flake or crack when rubbing it with the bare fingers, in accordance with EN 10244. The mesh wire shall show no rusty spots on any part of the surface excluding the cut ends. Minimum quantity of zinc (gm/sqm) based on the internal diameters of 2.2 mm, 2.7 mm & 3.4 mm should be 230, 245 and 265 respectively.

2.3.1.1.3 PVC coating

The initial properties of PVC coating material shall have a demonstrated ability to conform to the following requirements. The Specific Gravity should be in the range from 1.30 kg/dm^3 to 1.35 kg/dm^3 when tested in accordance with Test method ISO 1183. Tensile Strength should not less than 20.6 Mpa when tested in accordance with test method ISO 527. Elongation at break should not be less than 200% in accordance with ISO 527. The PVC coating shall not show cracks or breaks after the wires are twisted in the fabrication of the mesh.

Wherever, there is high changes of corrosion, alternate wetting and drying, high salinity, presence of shingles in water, a further refinement in coating shall be used like Galmac (where Zinc + 10% Aluminum) coating is applied to the main steel wire mesh. Further, if there is more severe condition, an additional coating of PVC coating shall be applied.

2.3.1.1.4 Mesh characteristics

Mesh wire: Diameter – Inner diameter shall be 2.7 mm for the Zinc coated wire and when measured with PVC coating the outer diameter shall be 3.7 mm.

Selvedge wire: Diameter – Inner diameter shall be 3.4 mm for the Zinc coated wire and when measured with PVC coating the outer diameter shall be 4.4 mm.

Mesh opening: Nominal Dimension D =100 mm.

Lacing and stiffener wire: Diameter – Inner diameter shall be 2.2 mm for the Zinc coated wire and when measured with PVC coating the outer diameter shall be 3.2 mm.

2.3.1.1.5 Boulders

The boulders for gabions shall be hard, angular to round, durable and of such quality that they shall not disintegrate on exposure to water or weathering during the life of the structure. Gabion rocks shall range between 0.15 m and 0.25 m. The range in sizes shall allow for a variation of 5% oversize and/or 5% undersize rock, provided it is not placed on the gabion exposed surface. The size shall be such that a minimum of three layers of rock must be achieved when filling the gabions of 1m thick.

2.3.1.2 Tolerances

Wire: Wire tolerances based on the internal diameters of 2.2 mm, 2.7 mm & 3.4 mm should be ± 0.06 mm, ± 0.06 mm and ± 0.07 mm respectively in accordance with EN 10218-2.

Mesh opening: Tolerances on the hexagonal, double twisted wire mesh, opening shall not exceed -4% to 16% on the nominal dimension value.

Gabions: 5 % (±) on the length, width, and height.

2.3.1.3 Various test for the gabions

Different tests to be carried on the gabion material are tabulated along with references and standards in Table 2-2.

Table 2-2: Various tests for the gabions

Mesh Type	*10' x 12'*	*Specifications*
Mesh Opening "D" mm	100	EN10223
Mesh Tolerance	+16% to –4%	EN10223
Unit Dimensions		L x W x H
Tolerances in sizes of units	± 5%	ASTM A975
Mesh Wire Diameter (mm)	2.7/3.7 (Inner Dia/Outer Dia)	EN10223
Tolerance (±) mm	0.08	B1052
Zn Coating Min (gsm)	240	ASTM A 641
Selvedge/Edge Wire Diameter (mm)	3.4/4.4 (Inner Dia/Outer Dia)	EN10223
Tolerance (±) mm	0.10	BS1052
Zn Coating (Selvedge/Edge Wire) Min (gsm)	260	ASTM A 641
Lacing Wire Diameter (mm)	2.2/3.2 (Inner Dia/Outer Dia)	
Tolerance (±) mm	0.06	BS1052
Zn Coating (Lacing Wire) Min (gsm)	220	ASTM A 641
Fasteners (mm)	3.0/4.0 (Inner Dia/Outer Dia)	
Stiffeners (mm)	2.2/3.2 (Inner Dia/Outer Dia)	
Zn coating on fastener/ stiffener (gsm)	240	ASTM A 641
PVC Coating		
Colour	Grey-RAL 7037	ASTM D 1482
Thickness Nominal (mm)	0.50	ASTM A 975
Thickness Minimum (mm)	0.38	ASTM A 975
Specific Gravity	1.30 – 1.35	ASTM D 792
Tensile strength	Not less than 20.6 MPa	ASTM D 412
Modulus of Elasticity	Not less than 18.6 MPa	ASTM D 412
Hardness	Between 50 and 60 Shore D	ASTM D 2240
Brittleness temperature	Not higher than –90C	ASTM D 746
Weight loss	Less than 5% after 24 hour at 1050 C	ASTM D 2287
Abrasion Resistance	The percentage of weight loss shall be less than 12%	ASTM D 1242
Salt spray Exposure and Ultraviolet Light exposure	a) The PVC shall show no effect after 3000 hours of salt spray exposure b) The PVC shall show no effect of exposure to ultraviolet light with test exposure of 3000 hours using apparatus Type E at 630C	ASTM B 117 ASTM D 1499 and G 23

	c) Evaluation of coating after salt spray and ultraviolet exposure test: After the salt spray test and exposure to ultraviolet light, the PVC coating shall not show cracks or noticeable change of colour, or blisters or splits. In addition, the specific gravity, tensile strength, hardness and resistance to abrasion shall not change more than 6%, 25%, 10% and 10% respectively from their initial values.	

2.3.2 Geo-textile bags

Geo-textile Bags are composed of woven/non-woven Polypropylene/Polyester Geo-textile. Double layer Geo-textile Bags using woven and non-woven geo-textile are used for harsh conditions. Geo-textile used to manufacture Geo-textile Bags should impart high mechanical properties for enhanced durability along with enhanced filtration, puncture, abrasion and U.V. resistance characteristics. Geo-textile should be inert to biological degradation and resistant to naturally encountered chemicals, alkalis, and acids.

Geo-textile used to manufacture Geo-textile Bags made of non-woven material shall conform to the properties listed in Table 2-3.

Table 2-3: Properties of non-woven geo-textile bag

Properties	*Test Method*	*Unit*	*Values*
Properties of Geo-textile			
Polymer Type			Polyester/PP
Nominal Mass	ISO 9864	Gms/Sq. m	≥400
Tensil Strength	ASTM D4595	kN/m (MD)	≥20
Tensile Strength	ASTM D4595	kN/m (CD)	≥20
Tensile Elongation	ASTM D4595	% (MD)	≥40% & ≤ 90%
		% (CD)	≥40% & ≤ 90%
CBR Puncture	IS 126	N	≥3900
Opening Size	ASTM D 4751	mm	≥0.07mm & ≤0.16mm
UV Reistance	ASTMD 4355	%/Hr	70/500
Properties of Geo-textile Bag			
Seam Type			Double Seam Chain stitch Twine of PP/PET
Preferable Lay Flat Dimensions			103 cm x 70 cm

Geo-textile used to manufacture Geo-textile Bags having double layers both for woven and non-woven material shall conform to the properties listed in Table 2-4.

Table 2-4: Properties of double layer geo-textile bag

Properties	*Test Method*	*Unit*	*Values*	
			Non Woven	*Woven*
Properties of Geo-textile				
Polymer Type			PP	PP
Weight	ISO 9864/ASTM D5261	Gms/Sq.m	≥300	≥230
Tensile Strength	ASTM D 4595	kN/m (MD)	≥12	≥35
		kN/m (CD)	≥12	≥35
Tensile Elongation	ASTM D 4595	% (MD)	≥30% & ≤90%	≥05% & ≤30%
		% (CD)	≥30% & ≤90%	≥05% & ≤30%
Grab Tensile Strength	ASTM D4632	kN	≥0.80	≥1.5
Grab Elongation	ASTM D4632	%	≥30% & ≤90	≥05% & ≤30%
Puncture Resistance	ASTM D4833	kN	≥.40	
Opening Size	ASTM D4751	mm	≥0.06 & ≤0.17	≥0.10 & ≤0.25
UV Resistance	ASTM D4355	%/hrs	70/500	70/500
Properties of Geo-textile Bag				
Seam Type			Double Seam Chain Stitch twine of PET /PP	
Preferably Flat Dimensions			2.00m x 1.50m	

Note: *Lay Flat dimensions of the Geo-textile Bags given in the table above are preferable sizes. The Client is free to use site specific sizes lesser than specified values but shall not exceed the dimensions given in the table.*

2.3.3 Geo-textile tubes

Geo-textile tube should be composed of high-tenacity polypropylene yarns which are woven into a rip-stop stable network such that the yarns retain their relative position. These Geo-textile Tubes are often filled hydraulically with slurry of sand and water, although many other fill materials have been used. The length of the Geo-textile tube should not be less than 10 m, but not to exceed 30 m, for the ease in placement and handling. The specifications of Geo-textile tubes differ for aggressive, non aggressive and very harsh coastal conditions. Geo-textile Tubes composed of Geo-composite consisting of double layer Geo-textile made of a polyester high-tenacity woven Geo-textile coupled to non woven polypropylene Geo-textile by needle punching method are suitable for very harsh coastal conditions. The Geo-textile tubes shall be constructed to meet the properties mentioned in Table 2-5 for aggressive condition.

Table 2-5: Properties of geo-textile tube for aggressive condition

Property	*Test Method*	*Units*	*Values*
Polymer	n/a	n/a	PP
Tube Circumference	Measured	m	4.3/8.6/12.9/14.2/16/17.2/21.6 ±5%
Fill Port (diameter)	Measured	cm	30 to 45
Tensile Strength (MD)	ASTM D4595	(kN/m)	≥175
Tensile Strength (CMD)	ASTM D4595	(kN/m)	≥175
Elongation - MD	ASTM D4595	%	≤16
Apparent Opening Size (AOS),O_{95}	ASTM D4751	(mm)	≤0.425 (No. 40 US Sieve)
Permittivity	ASTM D4491	1/S	≥0.4
Length		m	10/15/20/25/30

Table 2-6 contains properties of geo-textile tubes for non aggressive condition or typical condition.

Table 2-6: Properties of geo-textile tube for non-aggressive condition

Property	*Test Method*	*Units*	*Values*
Polymer	n/a	n/a	PP
Tube Circumference	Measured	m	4.3/8.6/12.9/14.2/16/17.2 ±5%
Fill Port (diameter)	Measured	cm	30 to 45
Tensile Strength (MD)	ASTM D4595	(kN/m)	≥70
Tensile Strength (CMD)	ASTM D4595	(kN/m)	≥105
Elongation - MD	ASTM D4595	%	≤16
Apparent Opening Size (AOS),O_{95}	ASTM D4751	(mm)	≤0.425
Permittivity	ASTM D4491	1/S	≥0.4
Length		m	10/15/20/25/30

Properties of geo-textile tubes for harsh conditions are given in and Table 2-7 below:

Table 2-7: Properties of geo-textile tube for very harsh condition

Property	*Test Method*	*Units*	*Value*
Polymer	n/a	n/a	PP Polyester
Circumference	Measured	m	4.3/8.6/12.9/14.2/16/17.2/21.6 ±5%
Fill Port (diameter)	Measured	cm	30 to 45

Tensile Strength (MD)	ISO 10319	(kN/m)	≥175
Tensile Strength (CMD)	ISO 10319	(kN/m)	≥175
Elongation at Max. strength-MD/CMD	ISO 10319	%	≤12/≤12
Opening Size (AOS),O_{90}	ISO 12956	μm	≤110
Permeability	ISO 11058	m/s	3×10^{-3}
Length		m	10/15/20/25/30

Note: *since the maximum circumferential stress is at the side of the filled Geo-textile, the seam position should be in such a way that, it does not coincides with the maximum stress position.*

2.3.4 Non woven geo-textile as filter

The nonwoven thermally bonded or needle punched or any equivalent geo-textile shall be used. The geo-textile shall be made of polyethylene or Polypropylene or polyester or similar fibers manufactured through machine made process of heat bonding or needle punching techniques. The material should have about 70% polypropylene and rest be polyethylene or any other equivalent material. The Minimum Average Roll Values of Geo-textile shall be as shown in Table 2-8.

Table 2-8: Properties of non-woven geo-textile as filter

#	*Properties*	*Mar Value*	*Test Method*
	Mechanical Properties		
1	Grab tensile strength	550 N	ASTM D 4632
2	Mullen Burst Strength	1000kPa	ASTM D 3786
3	Puncture Strength	200 N	ASTM D 4833
4	Trapezoidal tear strength	225 N	ASTM D 4532
	Hydraulic properties		
1	Apparent opening size	0.3 mm	ASTM D 4751
2	Permeability	0.1 cm/s	ASTM D 4491
	Physical		
	Unit Weight	>125g/sqm	

2.3.5 Vetiver for bank protection

The vetiver is a special type of grass having longer roots of length. This grass is infertile in nature. Due to their long roots and high tensile strength this grass is resistant to the high velocity streams and checks the erosion Desirable properties of the vetivers are given in Table 2-9.

Table 2-9: Properties of vetivers for bank protection

#	*Properties*	*Mar Value*
1	Average tensile strength	75 MPa
2	Root length	Up to 3 m
3	Life under 14 m of water	Up to 5 months
4	Air temperature range for sustainability	-14⁰C to 55⁰C
5	Soil Ph	3 to 10

2.4 References

Design practices and specifications adopted by the Maccafferi India Pvt Ltd.

Section-3: Design of Flood Embankment

3.1 General

A levee or dyke may be defined as an earthen embankment extending generally parallel to the river channel and designed to protect the area behind it from overflow of flood waters.

Embankments are the oldest known forms of flood protection works and have been used extensively for this purpose. These serve to prevent inundation, when the stream spills over its natural section, and safeguard lands, villages and other property against damages.

3.2 Planning of embankments

Basic understanding and steps to be taken up for planning of flood embankments has been described in the paras below:

3.2.1 Classification

Embankment Manual, CW&PC, 1960 stipulates that an embankment is designated as low, medium or major (according to its height above natural surface level (NSL). The details are as under in Table 3-1:

Table 3-1: Classification of embankment

#	*Classification of embankment*	*Criterion*
1	Low Embankment	Height < 10 ft (3 m)
2	Medium Embankment	10 ft (3 m) <Height> 30 ft (9 m)
3	Major Embankment	Height > 30 ft (9 m)

A typical high embankment with slope protection is shown in Figure 3-1.

Figure 3-1: Typical high embankment with slope protection using sand filled geo-mattress

3.2.2 Requirement of data

BIS code 12094: 200 stipulates that the following data is required for planning of an embankment.

3.2.2.1 Topographical data

Index plan showing area affected, contour survey plan of the area, past river courses, plan and section of earlier executed works.

3.2.2.2 Hydrological data

Discharge, gauge, velocity, carrying capacity, extent of spill of river, cross sections and longitudinal section of river, rainfall data for the basin sediment flow and river behavior like aggrading or degrading etc.

3.2.2.3 History of past floods

Indicating duration of floods, flood discharges and corresponding levels, stage of river at which damage was most pronounced, extent of damage etc.

Generally the data as mentioned above is not available while framing the detailed project report. The efforts should be made to collect the above mentioned data specially gauge and discharges past recent 10-20 years for the river from all possible sources.

3.2.3 Degree of protection

BIS code 12094: 200 stipulates that the height of embankment and the corresponding cost and Benefit Cost Ratio should be worked out for various flood frequencies taking into account the damages likely to occur. The degree of protection which gives the optimum Benefit Cost Ratio should be adopted.

Benefit Cost Ratio of the flood management/coastal protection scheme should be higher than 1. The B. C. Ratio is calculated dividing the Annual benefits of the scheme by annual cost of the scheme. The annual benefits should be based on losses for last 10 years duly verified by the Competent Authority. Annual cost of the scheme is taken as 16% of capital cost of scheme envisaging embankments.

Annual cost is adopted as 17% of capital cost of scheme envisaging anti erosion measures

However till such time as the details of all the relevant parameters are available, following criterion for degree of protection may be adopted.

3.2.3.1 Embankment for predominantly agricultural areas

The design flood for this type of embankment is kept 25 years for fixation of crest level.

3.2.3.2 Embankments

The design flood for this type of embankment is kept

for township or areas having industrial or other vital installations

100 years for fixation of crest level.

In the cases when anti erosion measures are proposed along with the embankment then design flood may be kept as 50 years for rural areas and 100 years for urban/industrial areas. In certain special cases, where damage potential justifies, maximum observed flood may also be considered for fixing the crest level.

However considering the change in hydrology due to effects of climate change, the return period may be adopted as 75 years and 150 years for rural and urban areas respectively *(However detailed discussion over adopting these higher values are required in later stage).*

In cases where Gauge & Discharge sites are not present, discharge may be worked out using the Empirical formula using the catchment area, extent of rainfall, catchment characteristics etc. Further hydrological data may be adopted using the regional hydrological booklets/manuals, prepared by the Hydrological Studies Organization, CWC. A typical earthen embankment is shown in Figure 3-2.

Figure 3-2: A typical earthen embankment

3.2.4 Alignment & spacing of embankment

BIS code 12094: 200 stipulates following guidelines related with the alignment and spacing of the embankment.

3.2.4.1 Alignment

The embankments should be aligned on the edge of natural bank of the river, where land is high and soil available for the construction of embankments. The alignment should be such that important township, vital installations, properties, cropped area is well protected by the embankment The alignment should be such that high velocity flow is quite distant from the toe of embankment to avoid scouring of the same and if embankments' alignment is near the high velocity flow then slope and toe protection in the form of pitching along with launching apron using the boulders, geo-bags, sand filled geo-mattress may be given. RCC porcupine screens along the toe line may also be used to retard the flow to induce siltation and check scouring of the toe-line. Alignment should also be planned so that land acquisition is feasible and nor prolonged.

3.2.4.2 Spacing

The spacing of embankments along the jacked reach of the river should not be less than 3 times Lacey's wetted perimeter for the design flood discharge. The minimum distance of the embankment from the river bank and midstream of the river should be one times Lacey's wetted perimeter and 1.5 times Lacey's wetted perimeter [Lacey's wetted perimeter (P) =4.75 $(Q_{design})^{½}$] respectively.

In the tidal reach of the river, embankments should be constructed with the due regard to the effects on their effect on navigation requirements in the channel as embankments in such cases may reduce the tidal influx causing a reduction in available navigation depth.

Length of the embankment: The length of the embankment directly depends upon the alignment. However, it is to be ensured that both ends of the embankment are tied up to some high ground or existing highway or railway or any other embankment nearby conforming to the design height of the embankment.

3.3 Design of embankment

BIS code 12094: 2000 is used for design of the embankment.

3.3.1 Types

As per Embankment Manual, CW&PC, 1960 and Irrigation and Hydraulic structures by S. K. Garg, embankments can be classified into three types as given below:

3.3.1.1 Homogenous embankment

This is simplest type of earthen embankment and consists of a single material and is homogeneous throughout. Sometimes, a blanket of relatively impervious material (stone pitching) may be placed at river side. A purely homogenous section is used, when only one type of material is economically or locally available. Such sections are used for low heights.

A purely homogenous section poses problems of seepage, and huge sections are required to make it safe against piping, stability etc. Due to this, homogenous section is generally provided an internal drainage filter like horizontal filter so that Hydraulic gradient line (HGL) is confined to the section. The length of horizontal filter may be kept as 3 times the height of dam or 25% to 100% distance of the toe from centre line of embankment.

Rock toe or Toe filter of (height = 25% to 35% of water height) consisting of fine sand, coarse sand and gravel, as per filter criterion requirement, may also be provided to check the seepage.

3.3.1.2 Zoned embankment

It consists of an inner core or section which is impervious section/core. The inner core checks the HGL. The transition zone prevents piping through cracks which may develop in the core. The outer zone gives stability to the central impervious core and also distributes the load over a larger area of foundations.

The core of the embankment may be constructed using the clay mixed with the fine sand or fine gravel. Pure clay is not best material for core as it shrinks and swells too much. Silts or silty clay may also be used as core.

Sand filled geo-tube may also be used as core of the embankment. Typical zoned embankment is shown in Figure 3-3.

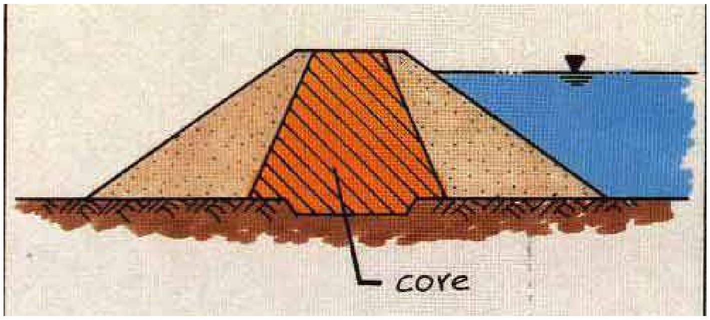


Figure 3-3: Embankment with core

3.3.1.3 Diaphragm type embankment

Diaphragm type Embankment has a thin impervious core, which is surrounded by the sand. The impervious core, called as diaphragm is made of imperious soils, concrete, steel, timber or any other material. It acts as water barrier to prevent seepage. The diaphragm must be tied to the bed rock or to a very impervious foundation material.

The diaphragm type of embankment is differentiated from zones embankment, depending upon thickness of core. If thickness of diaphragm is less than 10 m or less than height of embankment, the embankment is to be considered as diaphragm type embankment.

Sand filled geo-tube may also be used as core or diaphragm of the embankment.

3.3.2 Design HFL

The essential requirements for the design of embankment are the determination of design High Flood Level (HFL), hydraulic gradient, free board, side slopes, top width etc.

The design HFL should be obtained from gauge discharge relationship (G-D curve). The design HFL so obtained should be verified on the basis of observed cross-sections, slopes and velocities of the river in recent years. The design HFL may also be obtained from flood frequency analysis for corresponding return period.

When no discharge and gauge data is available, synthetic unit hydrograph approach should be used for estimating the desired return period flood. In some cases historically maximum observed flood level may also be considered as design HFL.

In the cases of embankments on both sides of river, rise in the water level due to jacketing of river should be kept in view in determining the design HFL.

3.3.3 Free board

The top of the embankment should be so fixed that there is no danger of overtopping, even with the intense wave wash or any other unexpected rise in water level due to sudden change in river course or aggradations of river bed or settlement of embankment.

The waves are generated on the surface by the blowing winds. height of water wave mainly depends upon the wind velocity (V in km/hr) and fetch or straight length (F) of water expanse in km. Wave height may be calculated using the equation given below:

h_w (in m) = 0.032 (V.F) $^{1/2}$+0.763-0.271 (F) $^{1/4}$ for F< 32 km

h_w (in m) = 0.032 (V.F) $^{1/2}$ for F> 32 km

The freeboard for wave action may be taken as 1.5x wave height (hw).

However in the absence of the wave data, free board should be taken as 1.5 for discharges less than 3000 cumec and 1.8 m for discharges more than 3000 cumec.

The free board proposed above as 1.5 m /1.8 m is recommended in case of less reliable and short-duration hydrological data to take care of uncertainty. In case hydrological data is collected using the reliable sources and length of such data is sufficient long like 35 years, then lesser values of freeboard like 1.0m/1.5m may be adopted.

It is also suggested to work out the maximum discharge corresponding to the crest level (adding the free board to the deign HFL). So that this maximum discharge can be compared with the higher return period discharges like SPF, PMF etc.

3.3.4 Top width

The top width of the embankment should be sufficiently enough for to accommodate the vehicular traffic. The top width of the embankment may be kept as 5.0 m. the turning platform may be kept 15 m to 30 m. An embankment should be provided with suitable soling over filter for proper drainage. For embankments protecting towns, industrial and vital installations, the necessity of providing all weather roads of 3 m to 3.5 m width should be examined. A typical crest of the embankment with brick soling at top is shown in Figure 3-4.

Figure 3-4: An embankment crest with brick soling

3.3.5 Hydraulic gradient

It is desirable to know the approximated line of seepage or hydraulic gradient line (HGL). The following guidelines may be used for determining the HGL:

Clayey soil: 4H:1V
Clayey sand: 5H:1V

Sandy soil: 6H:1V

3.3.6 Side slope

The side slopes are dependent upon the material and height of the embankment. The side slope should be flatter than the angle of repose of the material of the embankment. For drainage, longitudinal drains should be provided on the berm and cross drains at suitable places should be provided to drain out the water. In order to provide communication from one side of embankment to another side, ramps at suitable places should be provided as per requirement. Slope of typical embankment is shown in Figure 3-5.

Figure 3-5: Slope of a typical embankment

3.3.6.1 River side slope

The river side (RS) slope may be kept as 2H:1V or 2.5H:1V up to the height of 6 m depending upon the type of slope protection. It is preferable to have the draining material on river side slope to take care of sudden draw down. In case of high or important embankment, slopes are protected by the stone pitching, concrete blocks or sand filled geo-mattress to protected against sudden drawdown or erosive action of river flow.

3.3.6.2 Country side slope

A minimum cover of 0.6 m over the HGL should be maintained. For embankment up to height of 4.5 m, the country side slope should be 2H:1V from the top up to the point where the cover over HGL is 0.6 m after which a berm of suitable width, with country side slope of 2H:1V from the end of the berm up to the ground level should be provided.

For embankment of height from 4.5 m to 6.0 m, the country side slope the country side slope should be 3H:1V from the top up to the point where the cover over HGL is 0.6 m after which a berm of suitable width, with country side slope of 3H:1V from the end of the berm up to the ground level should be provided.

For embankment of height more than 6 m, detailed design should be done. Typical cross section of an earthen embankment is shown as under as Figure 3-6.

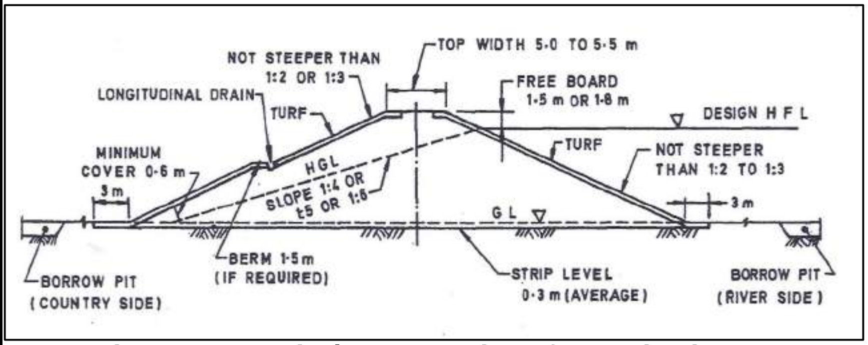


Figure 3-6: Typical cross-section of an embankment

3.3.7 Drainage

For drainage, longitudinal drains should be provided on the berm and cross drains at suitable places should be provided to drain the water from the longitudinal drains. Toe drain should be provided to prevent sloughing of toe. Perforated pipe embedded in properly designed graded filter with arrangements for disposal of water in the country side should be provided. Use of geo-textile material is also useful for safe drainage. Provision of drainage pipe and geo-textile material in embankment is shown in Figure 3-7 and Figure 3-8.

Figure 3-7: Provision of drain pipe

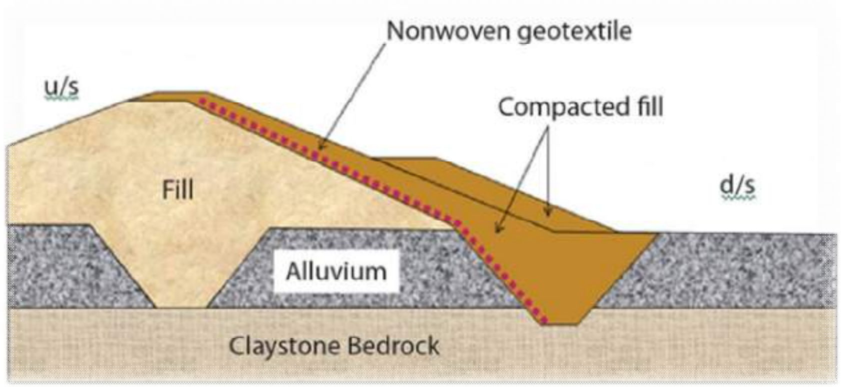


Figure 3-8: Provision of geo-textile material

3.3.8 Safety measures in design

Structure should be stable under all stages of construction and conditions of saturation and drawdown. It is therefore necessary that stability checks for various conditions should be done to ensure safety. Seismic forces should also be considered for high embankments. The factor of safety should be higher than 1.3.

Unequal settlements can be largely avoided by preparing the foundations properly and by selecting suitable material for construction. Whether the foundation soil is weak, suitable strengthening measures may be taken. Clayey soils containing organic matter such as remains of plants and root should be rejected. Well graded homogenous materials are most suitable for construction. In case of difficulty in getting full quantities of same material, zonal sections with impervious core and a pervious casing may be adopted. In high embankments it is desirable to mechanically compact the earth fill in suitable layers with a view to achieve optimum density with appropriate moisture content. Breaking of big clods specially in clayey soils is to be done ad organic/vegetable matter separated to safeguard against seepage/leakage/piping.

3.3.9 Sluices

Sluices with regulating arrangements should be provided for country side drainage. The size of sluice will depend upon the intensity of rainfall and the catchment area to be drained.

3.3.10 Causes of failure of embankment

As stipulated by the CBIP publication - 1989 River Behavior Management and Training Volume-I, in the absence of proper maintenance and supervision, embankments are susceptible to breaches due to various causes given below:

(a) Improper compaction and settlement of embankment.
(b) Transverse cracks due to unequal settlement.
(c) Inadequate drainage and pore pressure development.
(d) Erosion of riverside slope due to river current and wave wash
(e) Caving-in of the banks
(f) Overtopping of flood water during high floods.
(g) Failure of foundations due to infiltrations; and
(h) Piping as a result of insufficient cross-section. Leaks and cracks due to shrinkage of soil, and rat holes
(i) Increase in moisture content of the soil material.

3.3.11 Preventive measures

As stipulated by the CBIP publication- 1989 River Behavior Management and Training Volume-I, breaches/failures can be avoided by adopting suitable preventive measures mentioned briefly underneath:

(a) Toe drainage.
(b) Placing sands bags near toe (with drains covered by wooden planks) in order to increase shear resistance actuating forces causing slip
(c) Reducing seepage by lowering seepage head by constructing ring wells with sand bags near the toe
(d) Plugging piping holes with divers using tarpaulins soaked with bitumen from river side face of the hole
(e) Raising height of embankment (in case of overtopping) by using wooden planks without endangering stability against slip.

3.3.12 Closure of breach

Breaches should be closed on war footing so that the flooding of country side can be minimized. Usually retired embankments are constructed around the breach connecting the retired embankment with the original one at points sufficiently away from the breach so that by the time construction of retired embankment is over, the breach doesn't propagate to the connecting points. Vertical closure by driving piles all along the breach width is btter than trying to close the breach horizontally starting from ends, since here will be very high drag due to increasing velocity of the stream passing through the neck as the breach width gets narrowed down.

3.3.13 Protection of embankment

An embankment under direct attack of a river needs protection against failure. Different protective measures which are commonly employed to protect embankment are as under.

(a) Revetment/mattressing to protect against erosive action of river.
(b) Spurs/groynes to deflect dampen high velocity attacking the embankment

(c) Different grade control measures to tame a river flowing in steep terrain.
(d) Improving shear strength of embankment soil by growing shallow rooted vegetation.

A typical completed embankment is shown in Figure 3-9.

Figure 3-9: A typical completed embankment with R/S slope protection with mattress and turfing on C/S slope and road on crest

3.4 Stability analysis for high embankments

The most important cause of failure of an embankment is sliding. A portion of the earth may slide downwards and outwards with respect to remaining part, generally along a well defined slice surface. The failure is caused when the average shearing stress exceeds the average shearing resistance along the sliding surface due to various loading conditions.

Slope stability is generally analyzed by two methods depending upon the profile of failure surface viz. (a) Circular arc method and (b) Sliding Wedge method. In the 'Circular arc' method or 'Swedish Slip Circle' method, the rupture surface is assumed cylindrical or in the cross-section by an arc of a circle. The sliding wedge method assumes that the failure surface is approximated by a series of planes.

Generally for low embankments the sections designed on consideration of hydraulic gradient and cover are found to be quite suitable under all stages of construction, condition of saturation and draw down provided the foundation conditions are satisfactory. For high embankments the section proposed should be checked for stability by Swedish Circle method. The minimum factor of safety aimed at should be 1.3.

If the embankment is located in an area subjected to earthquakes, the forces developed by such movements should also be taken in to account while checking the stability. The value up to 0.1g for vertical acceleration and 0.2g for horizontal acceleration depending upon its distance from the epicenter may be assumed for the design purposes in areas subject to earthquake.

3.4.1 Selection of design parameters

The embankment material shear strength is obtained by performing triaxial tests of borrow area materials compacted to densities aimed at during construction. The foundation material strength is obtained by tests with undisturbed samples from tri-axial shear testing. Testing in each case shall be from zero to maximum normal stress expected in the embankment.

The design shear parameters for fill material is fixed at 75% availability from an adequate number of samples, and for foundation soils minimum shear strength values along foundation obtained are adopted after rejecting extreme or freak values.

3.4.2 Analysis procedure

The procedure of arriving at driving and resisting forces involves assumption of a tentative cross-section of the embankment, a possible circular failure surface, division of the slip circle mass into a number of slices, calculation of forces on each slice and summation of the forces. The factor of safety against sliding for assumed failure surface is obtained by the equation:

$$FS = \sum S / \sum V$$
$$= \Sigma[C + (N-U)\tan\phi] / \Sigma W \sin\alpha$$

Where:
FS = Factor of safety
S = Resisting or stabilizing Force
T = Driving or actuating force
C = C1x (b/ Cos α)
N = Force normal to the arc or slice
U = Pore water pressure.
ϕ = Angle of shearing resistance
W = Weight of the slice
A = Angle made by the radius of the failure surface with the vertical at the centre of slice.
c_1 = Unit cohesion, and b = Width of the slice

3.4.3 Stability computation

The slope stability analysis is carried out to get the minimum factor of safety for a tested section under different loading conditions for downstream and upstream slopes respectively. The computer programmes used for static analysis are used for the computations.

3.4.4 Final selection of embankment section

Based on the results of studies for slope stability by static and pseudo static method, the designer will select the final section of the embankment. In this selection, great emphasis is put on the experience of the designer and the data of behavior of embankments constructed in almost identical situations.

3.4.5 Defensive design measures

The design details should also include defensive measures to enhance their performance. The measures may include:

- Provision of adequate freeboard to allow for settlement, slumping and fault movement.
- Use of wide transition zones of materials not vulnerable to cracking.
- Use of drains near critical zones and central portion of embankment.
- Use of wide core zones.
- Use of adequate well-graded filter zone upstream of core to serve as a crack stopper.
- Controlled compaction of embankment zones.
- Removal or treatment of foundation materials that are of low strength or density.
- Widening of core at abutment interfaces.
- Special treatment of foundations at faults including provision of transition embankment sections.
- Stabilization of hill slopes susceptible to sliding around reservoir rim

3.5 Merits and demerits of embankments

Merits and demerits of flood embankments have been listed out below:

3.5.1 Merits

Merits of embankment as method of river training works are as under:

- Embankments are the main mean of preventing inundation during flood season.
- The initial cost of construction of embankment is low, although when raised subsequently, they become expensive.
- Construction is easy and presents no difficulty, as it can be done by utilizing local resources in unskilled labor and materials. Maintenance is equally simple and cheap.
- They can be executed in parts, provided that’s ends are properly protected.

3.5.2 Demerits

- Embankments cause raising of high flood levels.
- Embankments are fragile works. Bore holes caused works by small animals like crabs, rats and worms

may result in piping. They must be supervised closely during floods and protected, as soon as they are in danger.

- In the event of a breach, there is a sudden and considerable inflow of water which may cause damage in the country side and deposition of sand making the area infertile.
- Embankments are susceptible to direct attack of the river flow which can erode and undermine them.
- Low lying areas may become infested with water borne disease if proper drainage is not provided.
- In the case of river carrying considerable amount of silt, then deposition of silt on the river bed causes rise in the water level which may be lead subsequent overtopping of the crest level of embankment.

3.6 References

1. BIS code 12094:2000
2. Preliminary draft Guidelines for planning and design of river embankment (Levees) (Second revision of IS 12094) (Feb, 2011)
3. Embankment manual (1960)
4. Irrigation and Hydraulic structures- S. K. Garg
5. River Behavior Management and Training Volume-I (Central Irrigation and Power (CBIP), 1989

Section-4: Design of Bank Revetment

4.1 General

CBIP-manual "River Behavior Management and Training Volume-I -1989" stipulates that Protection of banks is a part and parcel of river training works because bank caving is one of the causes of deterioration of river conditions. River passing through populated/agricultural areas necessitates protection of adjacent lands and properties threatened by the erosion. The protection of river bank from the threat of erosion comes under Anti Erosion works. The purpose of bank protection may be training of river, protection of adjacent land and properties, protection of nearby hydraulic structures like embankments etc. Generally, bank protection works are auxiliary to river training works and expensive. Because of the high costs involved, all available materials are used. The river bank consists of the upper (above the Lowest water level or LWL) and lower sections (Below the LWL). The lower bank acts as the foundation for supporting the upper bank and,is, generally more susceptible to erosion. Recession of bank is caused by the erosion of lower bank, particularly at toe. The recession is fast, especially when there is sandy substrata below.

The upper bank is the portion between LWL and High Flood level (HFL). Action on this bank portion is most severe when the current impinges normal to the bank. During high stage of floods, erosion is also due to strong current along the bank. A typical bank protection work is shown in Figure 4-1.

Figure 4-1: A typical bank protection work

4.2 Causes of bank failures

CBIP-manual "River Behavior Management and Training Volume-I -1989" stipulates following causes of bank failure as listed below.

(a) Washing away the soil particles from the bank by strong current.
(b) Undermining the toe of bank by eddies, current etc followed by collapse of overhanging material deprived of support.
(c) Sliding or Sloughing of slope when saturated with water by floods of long duration. Saturation decrease the shear strength of soil. The stability is further reduced by the pressure of seepage flow.
(d) Piping in sub-layers due to movement of ground water towards the river, which carries away material with it.

Causes (a) and (b) of bank failures may be attributed to erosion. Failure under (c) is due to reduction in shear strength and under (d) results from foundation failure. An eroded bank is shown in Figure 4-2.

Figure 4-2: A typical eroded river bank

4.3 Methods of bank protection

CBIP-manual "River Behavior Management and Training Volume-I -1989" stipulates that bank protection works may be classified as direct and indirect. Direct protection works includes work done on the bank itself such as providing vegetal cover, pavement, revetment, grading of slope etc. Indirect protection of slopes includes works constructed not directly on the banks, but in front of them for reducing the erosive forces of the current, either by deflecting the current away from the banks or by inducing silt deposition near the bank.

If the current are not strong, banks can be protected by a vegetal cover using the shrubs and willows.

Sometimes a special vegetal cover like Vetiver (the long rooted grass having high tensile strength) can also be used for bank protection. In the case of strong current, banks may be protected by pitching using stone/boulders, concrete blocks, sand filled Geo-bags/Geo-mattress. The bank pitching is provided along with the launching apron to prevent the scouring under the water and the consequent fall of pitching.

In case of more steep slopes the banks can also be protected by retaining wall/guide walls to save the cost of cutting. This method is also beneficial when there is very limited land viz in hilly terrains.

Bank protection using the RCC porcupine screens comes under both type of method of protection viz. direct and indirect method. RCC porcupines are laid over the bank slopes and also protrudes into the river section to induce siltation near the bank. Some innovative methods foe bank protection are as under:

4.3.1 Vetivers

The vetiver is a special type of grass having longer roots of length up to 3 m. These roots have an average tensile strength of 75MPa. This type of grass is a perennial type of grass which can grow under extreme and wide type of conditions. This grass can live upto 5 months under 14 m of water. Due to their long roots and high tensile strength this grass is resistant to the high velocity streams and checks the erosion. The river bank before and after the implementation of vetiver is shown in Figure 4-3 and Figure 4-4.

Figure 4-3: Bank before the vetiver implementation

Figure 4-4: Bank after the vetiver implementation

4.3.2 Submerged vanes

Submerged vanes are under-water permeable type bank protection measures. These vanes are kept at an appropriate angle with the bank and at appropriate height with intention to induce optimum siltation near the bank. Orientation of submerged vanes should be decided after the model study to get their desired results. A typical cross section of river after implementing the submerged vanes is shown in Figure 4-5.

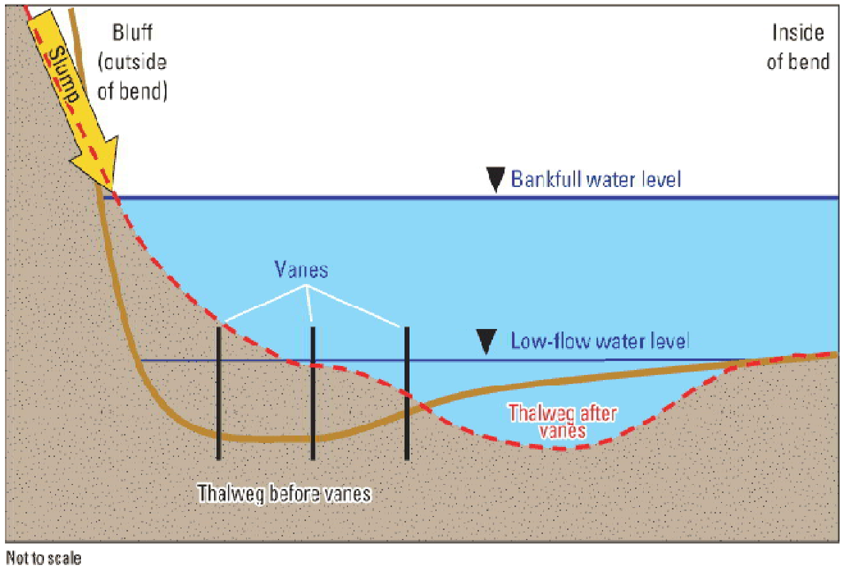


Figure 4-5: bank profile after implementing the submerged vanes

4.3.3 RCC kellener Jettys

RCC Kellner Jettys are monolithic RCC structures very similar to the RCC porcupines. RCC Kellner jettys are cast in-situ and consists of 3 RCC members. They are lighter than the RCC porcupines and there is no risk of rusting of the nuts and bolts as compared to the RCC porcupines. A typical picture of RCC kellener Jetty is given as Figure 4-6.

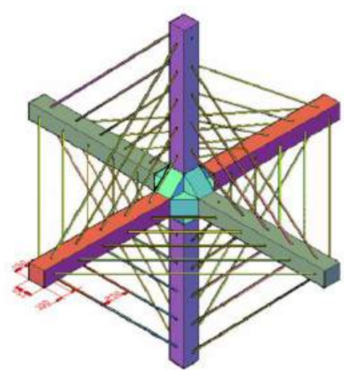


Figure 4-6: RCC kellenrr Jetty

4.3.4 Neoweb®

As per information from the website of PRS, the global leader in Neoweb®, the Neoweb® cellular confinement systems provide an ideal soil protection and soil stabilization solution by protecting the soil structure from erosive forces. The main difference between Neoweb soil stabilization and other geo-cells is the Neoloy® composite material. Neoloy provides outstanding dimensional stability and environmental durability. Neoweb guarantees reliable performance over the project lifespan due to Neoloy's:

- High dimensional stability under thermal cycling retains cell wall geometry over time
- High elastic modulus at elevated temperatures for reliable long-term performance
- High stabilizer content to protect from environmental oxidation
- High photochemical protection from UV light degradation

Neoweb®, earth stabilization provides effective protection with a variety of surface cover options: topsoil for landscaping; permeable aggregate for more protection where green landscape is not required or concrete hard-armor for more severe conditions.

Neoweb®, confinement ensures the long-term stability of slopes with various surface cover options: vegetated topsoil, gravel infill or concrete. The enhanced drainage, frictional forces and cell-soil-plant interaction of Neoweb®, prevents downslope movement:

- *Frictional resistance:* reduce sliding from hydrodynamic and gravitational forces.
- *Cell walls:* provide mechanical protection against run-off and rill development.
- *Confinement:* preserves integrity of soil structure to minimize impact of rain.
- *Perforations:* facilitates drainage and plant-root interlock.

The enhanced protection from Neoweb®, reduces the quantities of surface cover required–whether topsoil, gravel or concrete–making it cost-effective for almost every type of slope. In addition Neoloy® -based Neoweb®, offers long-term durability for the project lifespan. Neoweb®, is non-degradable and highly resistant to oxidation, UV radiation, and extreme temperatures. A typical Neoweb® is shown in Figure 4-7.

Figure 4-7: A typical Neoweb® preparation

4.4 Planning of bank revetment

IS code 14262:1995 mentioned following provisions regarding planning of bank revetment.

4.4.1 Requirement of data

Following data is required for planning of a bank revetment or pitching.

4.4.1.1 Topographical data

Index plan showing area affected, bank slope, type of soil, plan and section of earlier executed works.

4.4.1.2 Hydrological data

Discharge, gauge, velocity, cross sections and long section of river etc.

4.4.1.3 History of past erosion of banks during floods

Location of erosion prone sites, their extent, annual bank erosion lines, maximum annual erosion, annual erosion rate, extent of damage etc.

4.4.2 Degree of protection

The design flood for pitching/revetment may be calculated for 50 years return period using the flood frequency analysis. In certain special cases, where damage potential justifies, maximum observed flood may also be considered for fixing the crest level. The design HFL should be obtained from gauge discharge relationship (G-D curve).

In cases where G&D site are not present, discharge may be worked out using the Empirical formula using the catchment area, extent of rainfall, catchment characteristics etc. A typical bank pitching is shown in Figure 4-8.

Figure 4-8: A typical bank pitching in wire crates

4.5 Design of bank revetment

IS code 14262:1995 provides for following provisions regarding design of bank revetment.

4.5.1 Weight of stone/ boulder

Stone/boulders used in revetment for bank protection is subjected to hydrodynamic drag and lift forces. These destabilizing forces are expressed in terms of velocity, tractate forces etc. the stabilizing forces acting against these are component of submerged weight of stone and downward component of force caused by contact of the stones.

The weight of stones on slopes (W in kg) may be worked using the formula given below:

W (in kg) = $0.02323*Ss*V^6\ /K*\ (Ss-1)^3$ ----------------- (1)
Where K (correction factor for slope) =$[1-Sin^2\theta/Sin^2\Phi\ ]^{1/2}$
Ss=specific gravity of boulders ie 2.65
Φ = Angle of repose of material of protection works (adopted as 30^0 for boulders)
θ= Angle of sloping bank2 (H) :1 (V) (26.56^0)
V= Velocity in m/s
K =$[1-Sin^2 26.56^0/\ Sin^2 30^0]^{1/2}$ = =0.447
Hence weight of stones for 2H:1V slope
W (in kg) = $0.02323*Ss*V^6/0.447*\ (Ss-1)^3$

For river training works, sub-base is to be graded to a stable slope depending upon the angle of repose and cohesion of bank material under saturated condition and height of the bank. For high bank, berm may be provided. For important works, stability of bank with designed slope and berm should be checked by slip circle method or by soil dynamic testing procedures. For normal bank protection works, a slope of 2H:1V or flatter is recommended.

4.5.2 Size of stone/ boulder

Size of stone (Ds in m) may be determined from the following relationship.

Ds (in m) = 0.124* $(W/S_s)^{1/3}$ ------------------------------- (2)
Where W= Weight of stone in kg
S_s= Specific gravity of stone (adopted as 2.65)
Minimum diminution of stones > Ds

Generally, the size of stone should be such that its length, width and thickness are more or less same ie stones should be more or less cubical. Round stones or very flat stones having small thickness should be avoided.

4.5.3 Thickness of pitching

Minimum thickness of pitching (t) or protection layer is required to withstand the negative head created by the velocity. This may be determined by the following equation.

Minimum thickness of pitching (t in m) = $V^2/2g\,(S_s-1)$ -- (3)
V= Velocity in m/sec
g= Gravitational acceleration in m/sec2
S_s= Specific gravity of stone (Generally adopted as 2.65).

Therefore thickness of pitching should be higher than t (as obtained above). Two layers of stones of minimum size 't' should be provided for pitching in loose.

4.5.3.1 Pitching in crates

At high velocity, required weight of stones (to be found by Equation No 1) comes out to be higher, which makes handling and placing of stones a bit difficult. In such cases or in case when requisite sized stones are not available, small size stones filled in G. I. (Galvanized Iron) wire crates/nylon wire ropes (of adequate strength and with double knots and closely knit) may be used for pitching purpose. In this case single layer of G. I wire crates/nylon wire ropes crates filled with stones having thickness more than 't' may be used as pitching. The specific gravity of the crate is different from the boulders due to presence of voids. Porosity of the crates (e) may be worked out using the following formula.

E = 0.245+ 0.0864/ $(D_{50})^{0.21}$ ------------------------------- (4)
Where D_{50}= mean diameter of stones used in mm. let us assume D_{50} as 250 mm
e = 0.245 + 0.0864/ $(250)^{0.21}$
= 0.27

The opening in the wire net used for crates should not be larger than the smallest size of stone used. The mass specific gravity of protection (S_m) can be worked out using the following relationship.

$S_m = (1\text{-}e) * S_s$--- (5)

This mass specific gravity may be used to work out the weight of the crates and this weight should be more than weight of stone required, worked out by the equation No.1.

Crates should be laid with long dimension along the slope of the bank. Crates must be tied to each other by 5 mm GI wire/nylon rope as additional protection. If crates are being provided in layers then each layers should be tied to each other at suitable interval using the 4 mm GI wire/nylon rope.

4.5.4 Filter

A graded filter of size 150 mm to 300 mm thickness may be laid beneath the pitching to prevent failure by sucking action by high velocity. Geo-synthetic filter may also be used as they are easy to lay, durable, efficient and quality control is easy. A 150 mm thick sand layer over the Geo-synthetic filter may be laid to avoid its rupture of fabric by the stones.

4.5.5 Paneling

Paneling may be provided in the pitching where slope length is more so that slopes may remain more stable. The size of panel may be varied depending upon the length of river reach to be protected and the length of slope length. A typical bank pitching in crates is shown in Figure 4-9.

Figure 4-9: A typical bank pitching in crates

4.6 Design of typical bank revetment: an illustration

The design of typical bank revetment may be done as per following procedure.

Design Discharge Q (assumed)	=	20000	cumec
Gravitational Acceleration (g)	=	9.81	m/sec 2
Design HFL (Assumed)	=	100.00	m
Observed LWL (Assumed)	=	96.00	m
Stream Velocity V (Assumed)	=	3.00	m/sec
Mean Dia of river bed material d	=	0.30	mm

(assumed)			
Silt Factor f= 1.76* (d) $^{1/2}$	=	0.96	
Angle of sloping bank (2H:1V) θ	=	26.56	
Angle of repose of protection material Φ	=	30	
Value of $K=[1-\sin^2\theta/\sin^2\Phi]^{1/2}$	=	0.447	
Specific gravity of boulders Ss	=	2.65	
Weight of boulders $W=0.02323*Ss*V^6/(K*(Ss-1)^3)$	=	22.349	kg
Size of boulder= $0.124 (W/Ss)^{1/3}$	=	0.25	m
Thickness of pitching (T) for negative head criterion=V2/2g (Ss-1)	=	0.28	m
Thickness of pitching (=2*0.3=0.60m)	**=**	**0.60**	**m**
Design of Launching Apron (to be laid at LWL)			
Scour Depth below HFL D = $0.473*(Q/f)^{1/3}$	=	13.015	m
Max. Scour Depth below HFL due to bends etc (D_{max}) = 1.5*D	=	19.523	m
Width of Launching Apron = 1.5*[Dmax-(HFL-LWL)]	=	23.285	m
Adopt 16 crates of size 1.5mx1.5mx0.45 m (total width of launching apron=16*1.5=24)	=	24.000	m
Thickness of Launching Apron (2 layers of crates=2*0.45=0.90)) =1.5* 0.60	=	0.900	m
Size of Launching apron	**=**	**24x0.90**	m

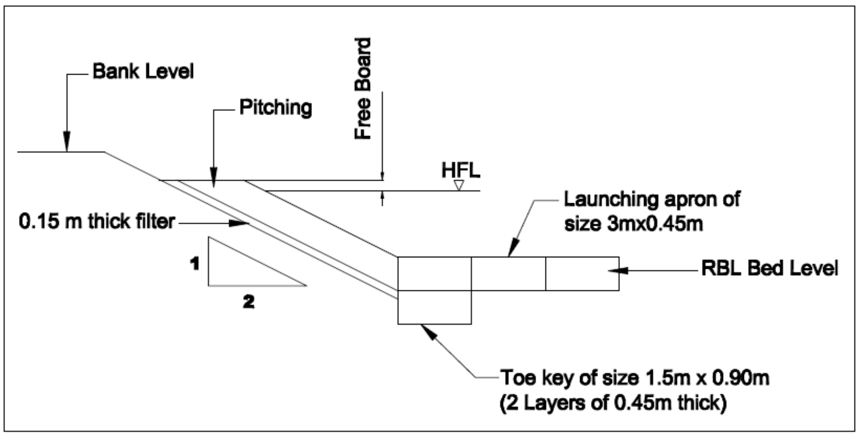


Figure 4-10: Typical cross-section of bank pitching with launching apron

4.7 Pitching in mortar

IS code 14262:1995 mentioned following provisions regarding pitching in mortar.

4.7.1 Size of stones

Stones, bricks or concrete blocks may be used for construction of pitching in mortar. Size of stones/bricks/concrete blocks in this type of pitching is not a critical aspect of design as every individual complement is bounded by mortar. Average size of available stone can be used for this purpose. But thickness of such pitching should be more than 't' (as calculated by the equation No. 3).

4.7.2 Paneling

Mortar revetment should not be constructed in continuous or monolithic form. To avoid cracks, joints at suitable interval may be provided. Generally revetment may be provided in panels of size 3mx3m or 3mx5m. The size of panel may be varied depending upon the length of river reach to be protected and the length of slope length. Standard stone filter or geo-synthetic filter may be provided beneath the joints.

4.7.3 Drain holes

Drain holes or weep holes may be provided in each panel for free drainage of pore water from saturated bank soil beneath it. Depending upon the size of panel, one or more weep holes may be provided for a panel. The pipe provided in the drain hole should be up to the natural bank. Stone graded filter or geo-synthetic filter may be provided at the end at the contact of the bank soil. Use of cement blocks in bank protection is shown in Figure 4-11.

Figure 4-11: Use of cement blocks in bank protection

4.8 Pitching in geo-textile bags

As per design practices following guidelines may be adopted for pitching in geo-textile bags.

4.8.1 Size of geo-bag

The pitching may also be provided in sand filled Geo-synthetic bags. The size of bags may be 1.1m x 0.7m x 0.15m. The weight of such bags is around 126 kg which is generally safe for the velocity up to 3 m/s. For higher velocities, size of Geo-bag may be higher so that weight of bag is higher than the require weight (worked out by the equation No.1. The Geo-synthetic material should be safe against the UV rays and abrasion.

4.8.2 Thickness of geo-bags pitching

The thickness of Geo-bag pitching may be decided as per procedure given above at 3.3. To summarize again, thickness of pitching should be more than ‘t’. Pitching may be provided in double layers of Geo-bags (in loose) and single layers if provided with the nylon ropes.

4.8.3 Filter

If the pitching is being provided in Geo-bags, then generally filter in not provided because material of Geo-bags itself work as filter. But for safety purpose (for taking care of bank soil in joints), a geo-synthetic filter layers beneath the Geo-bags may be provided. River bank before and after geo-bags for bank protection is shown in Figure 4-12 and Figure 4-13.

Figure 4-12: Bank before implementation of geo-bags

Figure 4-13: Bank after implementation of geo-bags

4.9 Toe protection

IS code 14262:1995 mentioned following provisions regarding toe protection.

To prevent the sliding and failure of the revetment on slope, toe is required to be protected. This may be in the form of simple toe-key, toe wall, sheet pile or launching apron.

4.9.1 Toe key

Simple key may be provided at the toe (may be called as toe key) when rock or un-erodible strata is available at the river bed and the overlaying banks are erodible. The key is in the form of stone/bricks or concrete blocks filled in the trench below the hard river bed for depth equal to the thickness of pitching “t” for proper anchorage. Sole purpose of this key is to provide lateral support to the pitching. The key may be of mortar or in geo-bags, if the pitching is provided in mortar or geo-bags.

4.9.2 Toe wall

When hard strata is available below the river bed at a reasonable depth, toe wall is recommended. The thickness of the toe wall depends upon height of wall and height of overlaying pitching. The toe wall may be design as retaining wall and be constructed in masonry along with provisions of weep holes etc.

4.9.3 Sheet piles and launching apron

When firm strata is not available at reasonable depth below the river bed, toe protection in the form of sheet pile or launching apron may be provided. The sheet pile may be made of RCC, steel or bamboo. The sheet piles may be drilled below the river bed up to maximum scour depth.

Sheet piles are difficult to drive, therefore Launching apron is preferred and provided with revetment. Launching apron should be laid at normal water level or low water level (LWL). The launching apron may be laid using the stones or Geo-bags. The stones/Geo-bags in the apron should be designed to launch along the slope of scour and provide a protection layer so that scouring is checked. The size of launching apron should be such that it should form a protection layer up to level of maximum scour depth. Slope of launching apron may be taken as 2H:1V. Filter below the launching apron may also be provided so that river bed material is safe against suction. Typical toe protection using toe wall and launching apron is shown in Figure 4-14.

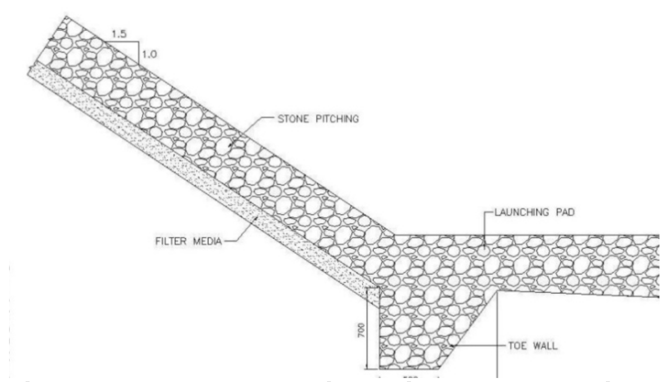


Figure 4-14: Toe protection suing toe wall and apron

4.9.4 Size of launching apron

Width of the launching apron depends upon the scour depth below HFL. Depth of scour below HFL (D) may be worked out suing the following formula:

$D = 0.473 (Q/f)^{1/3}$ --------------------------------------- (6.1)
and
$D = 1.33 (q^2/f)^{1/3}$ --------------------------------------- (6.2)
Where Q = design discharge in cumec and q = design discharge per unit width or design discharge intensity in cumec/m
f is silt factor. Silt factor (f) may be calculated using the following formula

$f = 1.76 (d)^{1/2}$ --- (7)
where d is mean particle diameter of river material in mm

Generally scour depth (D) below HFL should be calculated using the design discharge (Equation No.1). In some cases (for braided rivers) scour depth may be calculated using the design discharge intensity (equation No. 6.2).

Maximum scour depth (D_{max}) below HFL = 1.5* Scour depth (D below HFL).
Maximum Scour depth (D_{max}) below LWL = (D_{max}) below HFL – (HFL-LWL)
If the launching apron is being laid at LWL then width of the launching apron should be calculated using the following formula.
Width of launching apron= 1.5 * (D_{max}) below LWL
Thickness of launching apron (T) = 1.5* thickness of pitching (t).
In some cases, thickness of the launching apron is kept different from ‘T’ due to size of crates etc (if launching apron is being provided in crated stones), then width of the launching apron may be revised keeping the volume of stones/Geobag same per unit length of the apron.

4.10 Anchoring

IS code 14262:1995 mentioned following provisions regarding anchoring.

Proper anchor is required for keeping the revetment in place and serving the desired function. Upstream edge from where the revetment starts, should be secured well to the adjoining bank. Similarly, downstream edge where the revetment ends also needs to be secured well to the adjoining bank. Anchorage is also required to be provided on the top of submerged bank. If the top of bank is above HFL, the revetment should be provided above HFL with an adequate free board say 1.0 m. Under such situation, anchorage at top is not required.

4.11 Merits and demerits of bank protection

CBIP-manual “River Behavior Management and Training Volume-I -1989” stipulates that owing to its extent, bank protection is usually bulky and expensive. The failure of these type of works may be attributed to the over-economizing the cost of these type of works.

Cost can be reduced if protective measures are taken up immediately after signs of erosion of the banks. Otherwise deep scour develops, the river channel forms sharp and irregular bends and works become more expensive. Once a deep channel is developed, it is preferable to divert the current from the bank.

Cost of river training works vary widely from one river to another and it is very difficult to make a comparison. However for the protection of river bank and even already constructed embankment, bank protection works become necessity of the time.

The other demerit of these type of works may be termed as shifting of the erosion prone location from protected location to some other unprotected location in the d/s of the river

4.12 References

1. BIS code 14262:1995
2. Irrigation and Hydraulic structures- S. K. Garg.
3. River Behavior Management and Training Volume-I (Central Irrigation and Power (CBIP), 1989)
4. Draft for revision of IS code 14262:1995

Section-5: Design of Spurs/Groynes

5.1 General

CBIP-manual "River Behavior Management and Training Volume-I -1989" stipulates that protection of banks is a part and parcel of river training works. This protection comes under Anti Erosion works. Spurs/groynes are structures constructed transverse to the river flow and extended from the bank into the river. Spurs/groynes, protruding into river come under purview of Anti Erosion works. These type of works are provided to keep away the flow from the erosion prone bank. The spurs are provided along with the launching apron to prevent the scouring under the water and the consequent fall of spurs. Spurs/groynes can serve following functions:

- Training the river along a desired course by attracting, deflecting or repelling the flow in a channel;
- Creating a slack flow with the object of silting up the area in the vicinity;
- Protecting the river bank by keeping the flow away from it; and
- Contracting a wide river channel, usually for the improvement of depth for navigation.

A typical spur is shown in Figure 5-1.

Figure 5-1: Spur built with boulders

5.2 General design features

IS code 8408:1994 mentioned following provisions regarding planning of spurs.

5.2.1 Alignment

Spurs may be aligned either normal to flow direction or at angle pointing towards u/s or d/s of the flow. A spurs pointing u/s of the flow repels the flow away from the bank and is known as repelling type spurs/groyne. When a short length spur changes only direction of flow without repelling, it is known as deflecting spur/groyne. Spur pointing d/s of the flow attracts the flow towards the bank and is known as attracting spur/groyne. Generally repelling type or deflecting spurs are provided for anti erosion measures. Alignment of spurs at bend is shown in Figure 5-2.

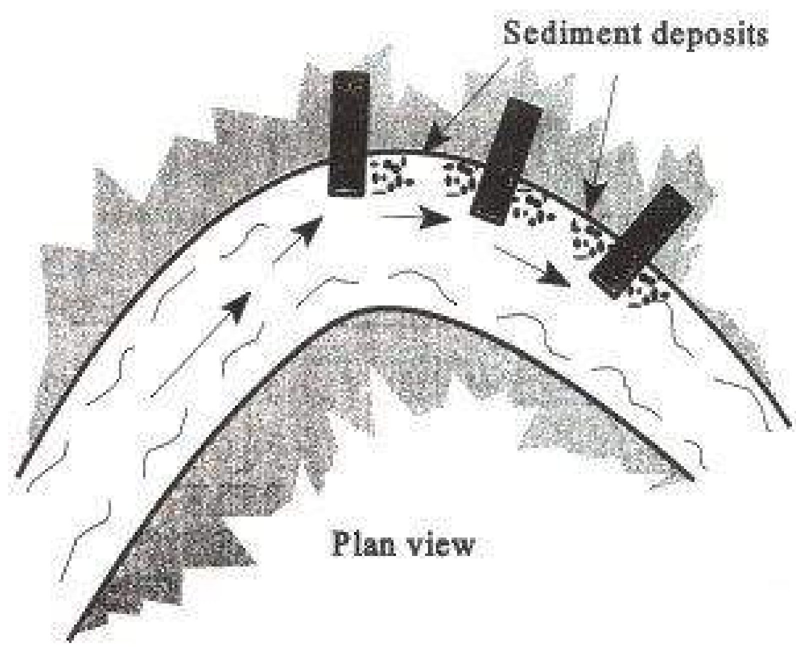


Figure 5-2: Alignment of spurs at bends to induce siltation

5.2.2 Functions of spurs

Spurs serve following functions:

(a) Training the river along the desired course to reduce the concentration of flow at the point of attack.
(b) Protecting the bank by keeping the flow away from it.
(c) Creating a slack flow with the object of silting up the area in the vicinity of the river bank.
(d) Improving the depths for navigation purpose.

5.2.3 Classification of spurs

These can be classified as follows, accordingly to;

(a) The methods and material of construction, namely permeable, impermeable and slotted.
(b) Height of spur with respect to water level, namely submerged, non-submerged and partially submerged (sloppy).
(c) Action, namely attracting or repelling.
(d) Special shapes, namely T-headed, hockey type or Burma type, kinked type

Impermeable or solid spurs are constructed with earth or rock-fill. Nose and adjacent shank portion is protected by heavy materials like stones in crates or concrete blocks.

In case of rivers carrying considerable amount of silt, permeable spurs are preferred. The permeable spurs offers flexibility in construction and maintenance and any alternation in later stage, if required, can be made. Common construction material for these type spurs is ballies, tress, bamboos and RCC porcupines. Generally RCC porcupine spurs are preferred as these are easy to construct, more durable and give better performance. Permeable spurs made with timber are shown in Figure 5-3.

Figure 5-3: Permeable spurs constructed with timber

5.2.4 Orientation of spurs

IS code 8408:1994 & *Draft for revision of IS 8408:1994* mentioned following provisions regarding orientation of spurs.

Spurs can be used single or in series, depending upon the reach of length to be protected. It can be used in combination with other river training measures. The spacing, orientation and length of spurs may be decided by the model study. Figure 5-4 illustrates a typical orientation of groynes field for training a river.

Figure 5-4: Groynes field for training a river

5.3 Design of boulder spurs

IS code 8408:1994 mentioned following provisions regarding design of spurs.

5.3.1 Length and spacing

The length of groyne should be decided on the basis of availability of land on the bank. Length shouldn't be less than that required to keep the scour hole formed at the nose away from the bank. Thus assuming angle of repose of sand to be 2.5H:1V and anticipated maximum scour depth below river bed (d_S), the length should be more than 2.5xd_S.short length may lead to bank erosion at u/s and d/s of the groyne due to formation of eddies at nose. On the other hand, too long groyne may construct the river and may not withstand the attack on account of heavy discharge concentration at the nose.

Normally the effective length of groyne shouldn't exceed 1/5th of width of flow in case of single channel. In case of wide, shallow and braided rivers, the protrusion of groyne in the deep channel should not exceed 1/5th of the width of channel on which the groyne is proposed excluding the length over then bank.

The spacing of groynes is normally 2 to 2.5 times its effective length. For site specific cases model studies may be conducted. The spurs located with proper spacing and length are shown in Figure 5-5.

Figure 5-5: Location of boulder spurs

5.3.2 Top level/top width and side slope

The top level of groyne will depend on the type namely, submerged, partially submerged or non-submerged and will be best decided by model study. In case of non-submerged groynes, the top level should be above design flood level with adequate free board. Free board may be adopted as 1m/1.5m. In case non-submerged groynes are tied with the embankment, then top level of embankment and top level of groyne may be kept same with similar free board and design HFL.

The top width of groyne should be 3 to 6 m as per requirement. Side slopes of the groyne may be kept 2H:1V or 3H:1V depending upon the material being used for construction.

5.3.3 Weight of stones for pitching

Stone/boulders used in pitching is subjected to hydrodynamic drag and lift forces. These destabilizing forces are expressed in terms of velocity, tractate forces etc. the stabilizing forces acting against these are component of submerged weight of stone and downward component of force caused by contact of the stones.

The weight of stones on slopes (W in kg) may be worked using the formula given below:

W (in kg) = $0.02323*Ss*V^6 / K* (Ss-1)^3$ ----------------- (1)
Where K (correction factor for slope) =$[1-Sin^2\theta/Sin^2\Phi]^{1/2}$
Ss = specific gravity of boulders ie 2.65
Φ = Angle of repose of material of protection works (adopted as 30^0 for boulders)
Θ = Angle of sloping bank 2 (H) :1 (V) (26.56^0)
V = Velocity in m/s
K = $[1-Sin^2 26.56^0 / Sin^2 30^0]^{1/2}$ = 0.447
Hence weight of stones for 2H:1V slope
W (in kg) = $0.02323*Ss*V^6/0.447* (Ss-1)^3$

5.3.3.1 Size of Stone/ boulder

Size of stone (Ds in m) may be determined from the following relationship.

Ds (in m) = $0.124* (W/S_s)^{1/3}$ ------------------------------- (2)
Where:

W = Weight of stone in kg
S_s = Specific gravity of stone (adopted as 2.65)
Minimum diminution of stones > Ds

Generally, the size of stone should be such that its length, width and thickness are more or less same ie stones should be more or less cubical. Round stones or very flat stones having small thickness should be avoided.

5.3.4 Thickness of pitching

Minimum thickness of pitching (t) or protection layer is required to withstand the negative head created by the velocity. This may be determined by the following equation.

Minimum thickness of pitching (t in m) = $V^2/2g\,(S_s-1)$ -- (3)
V= Velocity in m/sec
g= Gravitational acceleration in m/sec2
S_s= Specific gravity of stone (Generally adopted as 2.65).

Therefore thickness of pitching should be higher than t (as obtained above). Two layers of stones of minimum size 't' should be provided for pitching in loose.

5.3.4.1 Pitching in crates

At high velocity, required weight of stones (to be found by Equation No 1) comes out to be higher, which makes handling and placing of stones a bit difficult. In such cases or in case when requisite sized stones are not available, small size stones filled in G. I. (Galvanized Iron) wire crates/nylon wire ropes (of adequate strength and with double knots and closely knit) may be used for pitching purpose. In this case single layer of G. I wire crates/nylon wire ropes crates filled with stones having thickness more than 't' may be used as pitching. The specific gravity of the crate is different from the boulders due to presence of voids. Porosity of the crates (e) may be worked out using the following formula.

$e = 0.245 + 0.0864/(D_{50})^{0.21}$ ------------------------------- (4)
Where D_{50} = mean diameter of stones used in mm. let us assume D_{50} as 250 mm
$e = 0.245 + 0.0864/(250)^{0.21}$
= 0.27

The opening in the wire net used for crates should not be larger than the smallest size of stone used. The mass specific gravity of protection (S_m) can be worked out using the following relationship.

$S_m = (1-e) * S_s$-- (5)

This mass specific gravity may be used to work out the weight of the crates and this weight should be more than weight of stone required, worked out by the equation No.1.

Crates should be laid with long dimension along the slope of the bank. Crates must be tied to each other by 5 mm GI wire/nylon rope as additional protection. If crates are being provided in layers then each layers should be tied to each other at suitable interval using the 4 mm GI wire/nylon rope.

5.3.5 Filter

A graded filter of size 150 mm to 300 mm thickness may be laid beneath the pitching to prevent failure by sucking action by high velocity. Geo-synthetic filter may also be used as they are easy to lay, durable, efficient and quality control is easy. A 150 mm thick sand layer over the Geo-synthetic filter may be laid to avoid its rupture of fabric by the stones.

5.4 Pitching in mortar

IS code 14262:1995 mentioned following provisions regarding pitching in mortar.

5.4.1 Size of stones

Stones, bricks or concrete blocks may be used for construction of pitching in mortar. Size of stones/bricks/concrete blocks in this type of pitching is not a critical aspect of design as every individual complement is bounded by mortar. Average size of available stone can be used for this purpose. But thickness of such pitching should be more than ‘t’ (as calculated by the equation No. 3).

5.4.2 Drain holes

Drain holes or weep holes may be provided for free drainage of pore water from saturated bank soil beneath it. The pipe provided in the drain hole should be up to the natural bank. Stone graded filter or geo-synthetic filter may be provided at the end at the contact of the bank soil.

5.5 Pitching in geo-bags

As per design practices following guidelines may be adopted for pitching in geo-textile bags.

5.5.1 Size of geo-bag

The pitching may also be provided in sand filled Geo-synthetic bags. The size of bags may be 1.1 mx0.7mx0.15 m. The weight of such bags is around 126 kg which is generally safe for the velocity up to 3 m/s. For higher velocities, size of Geo-bag may be higher so that weight of bag is higher than the require weight (worked out by the equation No. 1. The Geo-synthetic material should be safe against the UV rays and abrasion.

5.5.2 Thickness of geo-bags pitching

The thickness of Geo-bag pitching may be decided as per procedure given above at 3.3. To summarize again, thickness of pitching should be more than ‘t’. Pitching may be provided in double layers of Geo-bags (in loose) and single layers if provided with the nylon ropes.

5.5.3 Filter

If the pitching is being provided in Geo-bags, then generally filter in not provided because material of Geo-bags itself work as filter. But for safety purpose (for taking care of bank soil in joints), a geo-synthetic filter layers beneath the Geo-bags may be provided. A spur using the geo-bags is shown in Figure 5-6:

Figure 5-6: Spur using geo-bags

5.6 Launching apron for spur

IS code 8402:1994 & 14262:1995 mentioned following provisions regarding pitching in mortar.

To prevent the sliding and failure of the groyne due to scouring action by the river current, provision of launching apron is kept to take care of the scouring at nose and at shank (portion in the river) of the groyne.

Launching apron should be laid at normal water level or low water level (LWL). The launching apron may be laid using the stones or Geo-bags. The stones/Geo-bags in the apron should be designed to launch along the slope of scour and provide a protection layer so that scouring is checked. The size of launching apron should be such that it should form a protection layer up to level of maximum scour depth. Slope of launching apron may be taken as 2H:1V. Filter below the launching apron may also be provided so that river bed material is safe against suction.

5.6.1 Size of launching apron

Width of the Launching apron depends upon the scour depth below HFL. Depth of scour below HFL (D) may be worked out suing the following formula.

$D = 0.473\ (Q/f)^{1/3}$ ------------------------------------- (6.1)
and
$D = 1.33\ (q^2/f)^{1/3}$ -------------------------------------- (6.2)
Where Q= design discharge in cumec and q= design discharge per unit width or design discharge intensity in cumec/m.
f is silt factor. Silt factor (f) may be calculated using the following formula
$f = 1.76\ (d)^{1/2}$--- (7)
where d is mean particle diameter of river material in mm

Generally scour depth (D) below HFL should be calculated using the design discharge (Equation No.1). In some cases (for braided rivers) scour depth may be calculated using the design discharge intensity (equation No. 6.2).

Maximum scour depth (D_{max}) below HFL= 1.5* Scour depth ($D_{below\ HFL}$).
Maximum Scour depth (D_{max}) below LWL = $(D_{max})_{below\ HFL}$ – (HFL-LWL)

If the launching apron is being laid at LWL then width of the launching apron should be calculated using the following formula at different locations of the groyne.

(i)	Width of launching apron at nose	=	(2-2.5) * $(D_{max})_{below\ LWL}$
(ii)	Width of launching apron at transition from nose to shank and first 30 m to 60 m in u/s	=	1.5 * $(D_{max})_{below\ LWL}$
(iii)	Width of launching apron in shank portion for next 30 m to 60 m	=	1.0 * $(D_{max})_{below\ LWL}$
(iv)	Width of launching apron at transition from nose to shank and first 15 m to 30 m in d/s	=	1.0 * $(D_{max})_{below\ LWL}$

Thickness of launching apron (T) = 1.5* thickness of pitching (t).
In some cases, thickness of the launching apron is kept different from 'T' due to size of crates etc (if launching apron is being provided in crated stones), then width of the launching apron may be revised keeping the volume of stones/Geobag same per unit length of the apron.

Figure 5-7: Spur along with bank protection using geo-textile

5.7 Design of typical boulder spur: an illustration

The design of typical boulder spur may be done as per following procedure

Design Discharge Q (assumed)	=	20000	cumec
Gravitational Acceleration g	=	9.81	m/sec2
Design HFL (Assumed)	=	100.00	m
Observed LWL (Assumed)	=	96.00	m
Stream Velocity V (Assumed)	=	3.00	m/sec
Mean Dia of river bed material d (assumed)	=	0.30	mm
Silt Factor f= 1.76* (d) $^{1/2}$	=	0.96	
Angle of sloping bank (2H:1V) θ	=	26.56	^{0}
Angle of repose of protection material Φ	=	30	^{0}
Value of K=$[1-\sin^2\theta/\sin^2\Phi]^{1/2}$	=	0.447	
Specific gravity of boulders Ss	=	2.65	
Weight of boulders W=0.02323*Ss*V^6/(K* (Ss-1) ^{3})	=	22.349	kg
Size of boulder= 0.124 (W/Ss) $^{1/3}$	=	0.25	m
Thickness of pitching (T) for negative head criterion=V2/2g (Ss-1)	=	0.28	m
Thickness of pitching (=2*0.3=0.60m)	=	**0.60**	**m**
Design of Launching Apron (to be laid at LWL)			
Scour Depth below HFL D = 0.473*(Q/f) $^{1/3}$	=	13.015	m
Max. Scour Depth below HFL at Nose ($D_{max)}$ = (2.0-2.5) *D (adopted as 2D)	=	26.030	m
Max. Scour Depth below HFL at transition from nose to shank and 1ST 30 m to 60 m U/S (D'max) = 1.5*D	=	19.523	m
Max. Scour Depth below HFL for next 30 m to 60 m in U/S (D''max) = 1.0*D	=	13.015	m
Max. Scour Depth below HFL for	=	13.015	m

transition from nose to shank and 1ST 15 m to 30 m D/S (D'''max) = 1.0*D			
Width of Launching Apron at nose= 1.5*[Dmax- (HFL-LWL)]	=	33.045	m
Adopt 23 crates of size 1.5mx1.5mx0.45 m (total width of launching apron=23*1.5=34.5)	**=**	**34.50**	**m**
Width of Launching Apron for transition from nose to shank and up to 60-90 m U/S =1.5*[D'max- (HFL-LWL)]	=	23.285	m
Adopt 16 crates of size 1.5mx1.5mx0.45 m (total width of launching apron=16*1.5=24)	**=**	**24.00**	**m**
Width of Launching Apron for next 30 m to 60 m in U/S =1.0*[D''max- (HFL-LWL)]	=	9.015	m
Adopt 6 crates of size 1.5mx1.5mx0.45 m (total width of launching apron=6*1.5=9)	**=**	**9.000**	**m**
Width of Launching Apron for transition from nose to shank and 1st 15 m to 30 m D/S =1.0*[D'''max- (HFL-LWL)]	=	9.015	m
Adopt 6 crates of size 1.5mx1.5mx0.45 m (total width of launching apron=6*1.5=9)	**=**	**9.000**	**m**
Thickness of Launching Apron (loose boulder) =1.5* Thickness of pitching	**=**	**0.900**	**m**

5.8 Permeable spurs

Draft for 2nd revision of IS code 14262:1995 mentioned following provisions regarding permeable spurs.

5.8.1 Introduction

Unlike impermeable groynes which do not allow any water to flow through its body (except seepage due to differential head), permeable groynes made of ballies, bamboos, trees, porcupines etc. are pervious enough so that the flow takes place across the groynes through their bodies. As found experimentally (by Lagasse-), up to 35% permeability (defined as the area of opening to the total area of flow intercepted by groynes i.e. the product of its length normal to the flow and the depth of flow), the behavior of a permeable groyne, as far as its effectiveness in bank protection is concerned, is almost similar to that of an impermeable groyne. As the permeability increases, the length of the protected reach of bank gets reduced since the eddies are reduced. As the flow passes through the permeable groynes, the micro eddies and the turbulence produced downstream of the groynes cause dampening of flow (due to energy dissipation) and consequent reduction in velocity. As a result the erosive power of the flow is reduced.

Permeable groynes are usually made in groups and may be used in combination with an upstream impermeable groyne acting as flow deflector and sheltering the permeable groynes from any direct attack on them. They may also be constructed in combination with longitudinal dykes and revetments. Permeable groynes are less costly compared to impermeable ones and are usually made of locally available materials. They may be both submerged and un-submerged types and are preferred in meandering reaches with deep water near concave bank. Due to dampening of flow, the sediment carrying capacity of flow behind the groynes get reduced resulting in deposition of sediments and building of banks along the affected reach.

5.8.2 Classification of permeable spurs

The permeable groynes can be classified as follows:

(a) According to functions served, namely, diverting and dampening, sedimenting.
(b) According to materials used for construction.
(c) According to flow conditions, namely, bally, bamboo, tree, willow.
(d) According to forms, namely, Pile spurs, screens, porcupines, cribs, framed structures, trees, willows, tetrahedrons.

Different types of elements are used for making permeable types of groynes. The dimensions of the elements vary according to the sizes readily and commercially available in the market/locality. The different forms of permeable groynes are briefly described below.

5.8.2.1 Pile spurs/ groynes

These are driven piles made of shawl or bamboo in one or more rows and are tied together with stringers. They have also been extensively used in Mississippi and Missouri rivers in USA for controlling riverbank erosion. The driven length below bed will be governed by the extent of scour and pile stability. Riverbed and bank around these piles must be protected with stone pitching. The spacing between the consecutive piles or clumped piles is governed by desired permeability varies from 30 to 50 %. The length of spur (L) determined by numbers of pile as in a row is found from the desired and existing bank lines. The spacing between two consecutive rows of pile spurs is usually kept between 3 to 4 times their lengths.

5.8.2.2 Kellner Jetty/ porcupines

They are made of ballies or bamboos or RCC having cubical shaped box at the central portion with their legs extending in different directions. The central box is filled with stones for the stability of the individual units of porcupines having size varying from 2 to 3 m. The individual units are placed side by side in a row and are tied. The spacing between the two consecutive units of porcupines will depend upon the desired permeability varying from 30 to 50%. The spacing of two consecutive rows of porcupines varies from 3L to 4L, where L is the length of spur. A typical RCC porcupine spurs is shown in Figure 5-8.

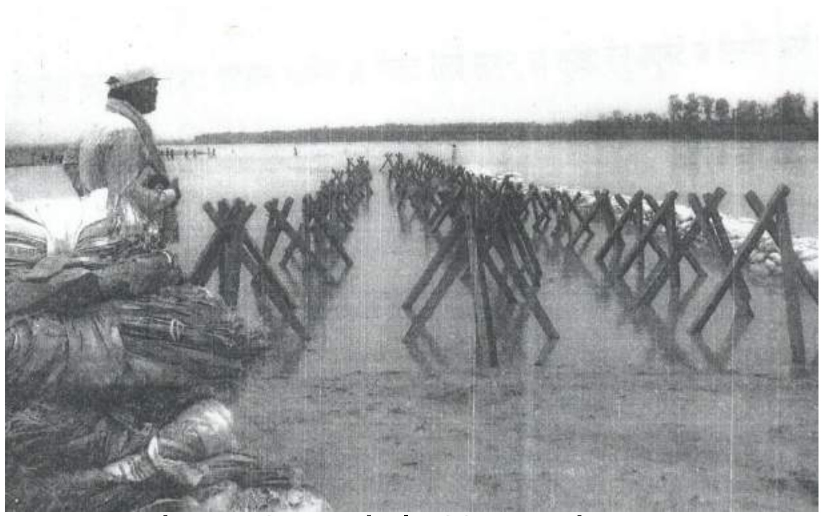


Figure 5-8: A typical RCC porcupine spur

5.8.2.3 Cribs

These are similar to porcupines with the difference that the ballies/bamboos from a pyramid type structure with a box at the bottom for holding stones for the stability of individual units. The spacing between the consecutive cribs and the consecutive rows of cribs will be similar to that of porcupines.

5.8.2.4 Balli/bamboo frames

A framed structure made with driven poles of bamboos/shawls with longitudinal, cross and diagonal bracings is constructed across the flow.

5.8.2.5 Tree spurs

Locally available tree branches (hung with their trunks up and branches/leaves down tied with ropes across a framed structure are found to be highly effective in dampening flow velocity and entrapping suspended sediments in the flowing water.

5.8.2.6 Willow/ brushwood spurs

Willow (also called as Tarza or Shirkanda) is a type of bush available in plenty across the country, has sufficient rigidity and strength are not easily decomposed. These or other brushwood available locally are filled and weighed by heavy stones in alternate layers within the framed structures. Such spurs however, entrap sediments and loose their initial permeability and eventually behave like impermeable spurs with deep scour near their noses.

5.8.3 Submergence of spurs

Unlike impermeable Spurs which are un-submerged with freeboard, Permeable spurs may be either un-submerged or submerged. Submergence up to 50% is acceptable for Porcupines, 20% for cribs and 5% to 10% for tree and willow spurs with framed structure.

5.8.4 Length and spacing of permeable spurs

Considerations similar to those as already discussed for impermeable groynes under clause 5.2.should be followed in deciding length and spacing of permeable spurs. Very long spurs should not be provided due to difficulties in construction as well as maintenance against scour. The spacing of spurs will be determined by their lengths. Shorter spurs at closer interval is desirable in curved outer banks of a meandering stream compared to those in the straight reach of rivers.

5.9 Limitation of spurs

CBIP-manual “River Behavior Management and Training Volume-I -1989” stipulates that the success of repelling type groyne depends upon the extent and the quickness with which scour occurs at the nose, and also on how quickly the pockets between the groynes get filled up with sediment. This condition make the impermeable groynes useless in boulder rivers, in which the rate of silt deposition may be slow or in flashy rivers in which floods rise and fall so quickly that desired silting doesn’t take place. The groynes can’t be relied upon to afford immediate protection.

It is also observed that silting between the successive groynes can be accomplished only when their lengths are sufficient. Short groynes don’t offer sufficient protection.

In the case of narrow and deep rivers, the cost of solid groynes above high water is substantial. Moreover, because of the narrow width of rivers, solid groynes can’t be extended much as otherwise they can cause harmful conditions on the opposite bank or further d/s. In such cases submerged groynes are recommended. As the tractive force on the slope is maximum at 1/3 depth from the bottom, the top of groyne should be kept at least half of depth of water. A single submerged groyne may not be as effective as series of submerged groynes. Since flow over the groynes produces turbulence and scour below them, silting may not take place as rapidly as required. If there are important structures or valuable properties on the bank, submerged groynes should be preferred since their action is not as violent as that of submerged solid groynes. It may be concluded that permeable spurs are effective only in rivers which carry heavy suspended load.

5.10 References

1. IS code 8408:1994.
2. Irrigation and Hydraulic structures- S. K. Garg.
3. River Behavior Management and Training Volume-I (Central Irrigation and Power (CBIP), 1989).
4. Draft for revision of IS code 8408:1994.

Section-6: Design of RCC Porcupine Screens/Spurs/Dampeners

6.1 General

Protection of banks is a part and parcel of river training works. This protection comes under Anti Erosion works. Permeable structures envisaging construction of RCC porcupine screens/spurs/ dampeners are a cost effective alternative to the impermeable bank protection works for the rivers carrying considerable amount of silt. RCC porcupine is a prismatic type permeable structure, comprises of six members of made of RCC, which are joined with the help of iron nuts and bolts.

Permeable screens, spurs, dampeners are the main type of permeable structures in vogue. Prima facie, the purpose, overall behavior and layout of the above mentioned structures can be compared to those of submersible bunds, groynes and revetment respectively. The permeable structures can be used either independently or with a support of other impermeable boulder structures or river training and bank protection measures. Depending upon the purpose, the permeable structures (RCC porcupines) may be constructed in transverse or parallel to direction of flow. Typical sketch of a RCC porcupine is as given at Figure 2-3.

6.2 General design features

General design features of RCC porcupine screens/spurs/ dampeners are given below in detail.

6.2.1 Concept

Dampening of velocity is achieved by using the permeable structures. If the flow is sediment laden, siltation is induced in the slack flow region and the channel is shifted away from the protected reach.

If the flow is not carrying the sufficient sediments, only dampening of velocity can result. Sedimentation may not be achieved near the protected reach.

Only partial obstruction to the flow of about 15 to 20% only is envisaged in the design. Higher obstruction causes more diversion of flow resulting in undesired scouring around the proposed structures, particularly at the nose portion. Additional protection to the nose and flanks is required to avoid such scour. Therefore, obstruction more than 20% is avoided.

Submergence of RCC porcupine screens/spurs/ dampeners is kept upto 50% of depth of flow. For example, single layer of RCC porcupines is sufficient for depth of water till 6 m.

6.2.2 Functions of permeable structures

Permeable structures serve one or more of the following functions:

(a) Training the river along the desired course.
(b) Reducing the intensity of flow at the point of river attack.
(c) Creating a slack flow to induce siltation in the vicinity of the permeable structures and in the d/s reach.
(d) Providing protection to the bank by dampening the velocity of flow along the bank.

Use of RCC porcupines for slackening the flow is shown in Figure 6-1.

Figure 6-1: RCC porcupines for slackening the flow

6.2.3 Structural elements

The elements used in the RCC porcupine screens/spurs are as under:

(a) Members/Elements: The porcupines are made of RCC members/elements. These members are casted in-situ at the site or location near the site. Generally six members are used to construct one porcupine. The size of one member is kept as 3mx0.1mx0.1m or 2mx0.1mx0.1m. These members are joined with the help of Nails.
(b) Nails: Standard commercially available nails of length 100 mm to 150 mm are used to join the porcupine members. Double nailing at critical joints may be provided.
(c) GI Wire: 4 to 5 strands of 4 mm GI wire should be sued for inter-connecting the porcupines and may be

anchored with the ground. Alternatively, 12 mm 3-4 strands wire ropes should be used for the interconnecting the porcupines.

Use of nylon ropes instead of GI wires may also be tried. But these are susceptible to disintegration in presence of UV rays. In view of this, UV stabilized nylon ropes may be tried instead of GI wires due to their better usability and flexibility and non-corrosive behavior. A typical RCC porcupine screen is shown in Figure 6-2.

Figure 6-2: Typical RCC porcupine screen

6.2.4 Layout in plan

As already pointed out that the RCC porcupine screens are laid in the form of screens, spurs and dampeners. The layout for each of the structure is as under:

6.2.4.1 The RCC porcupine spurs

(a) The porcupines (comprising of six members of size 3mx0.1mx0.1m) are laid in a row across the river bank protruding into the river at spacing generally adopted as 3m c/c. If size of member is 2m x 0.1m x 0.1m, then spacing between the porcupine may be kept as 2m c/c.

(b) Each porcupine spur is made up of 3 to 7 rows of porcupines (Higher rows for higher flow). The spacing of rows is kept at same as spacing of each porcupine in each row (3 m or 2 m c/c depending upon size of the member).

(c) If the flow depth is more than 6 m, RCC porcupine spurs may be provided in double vertical layers.

(d) On a straight reach, RCC porcupine spurs are placed at 3 to 5 times the length of spur. On a curved channel, the spacing can be kept as 2 to 4 times the length of spur.

(e) The length of spur into the river shouldn't exceed the 1/5 of the width of the flow. Generally length of spurs is less than 100m to 150 m.

(f) In order to resist the tendency of outflanking, additional porcupines may be provided along the sloping bank at u/s and d/s of the RCC porcupine spurs.

(g) At least three RCC porcupine spurs may be provided for a reach to be protected. A single permeable spur is generally not effective.

(h) The practice of providing one or two additional spurs u/s and d/s of the eroding reach, pointing towards u/s with reference to flow may be followed for the RCC porcupine spurs also. Typical sketch of RCC porcupine spur is as under in Figure 6-3.

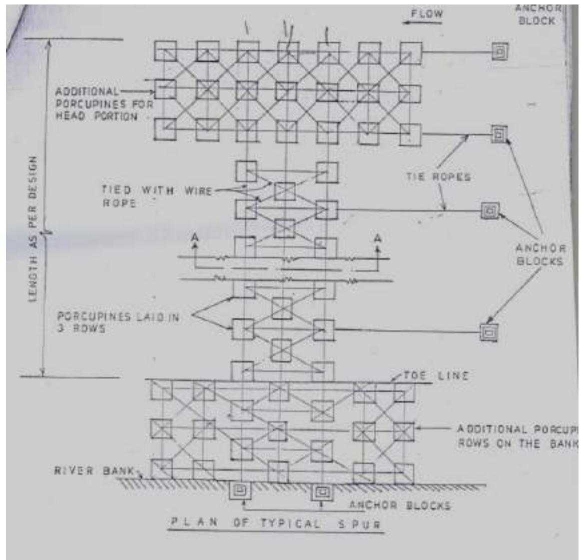


Figure 6-3: A typical porcupine spur

A RCC porcupine spur is also shown in Figure 6-4.

Figure 6-4: RCC porcupine spur

6.2.4.1 The RCC dampeners

(a) The porcupines (comprising of six members of size 3mx0.1mx0.1m) are laid in a row along the river bank functioning similar to revetment at spacing generally adopted as 3m c/c. If size of member is 2mx0.1mx0.1m, then spacing between the porcupine may be kept as 2m c/c.

(b) The spacing of rows is kept at same as spacing of each porcupine in each row (3 m or 2 m c/c depending upon size of the member).

(c) For a maximum depth of flow upto 3 m, 2 rows of porcupines are laid along the either side of toe as dampeners.

(d) For a depth of flow more than 3 m, rows of porcupines are added across the bank line up to HFL @ spacing of 3 m or 2 m depending upon the length of member. Additional rows (2 or 3) may be provided at the top of bank in case of submerged river bank during floods. Typical sketch of RCC porcupine dampener is as under in Figure 6-5.

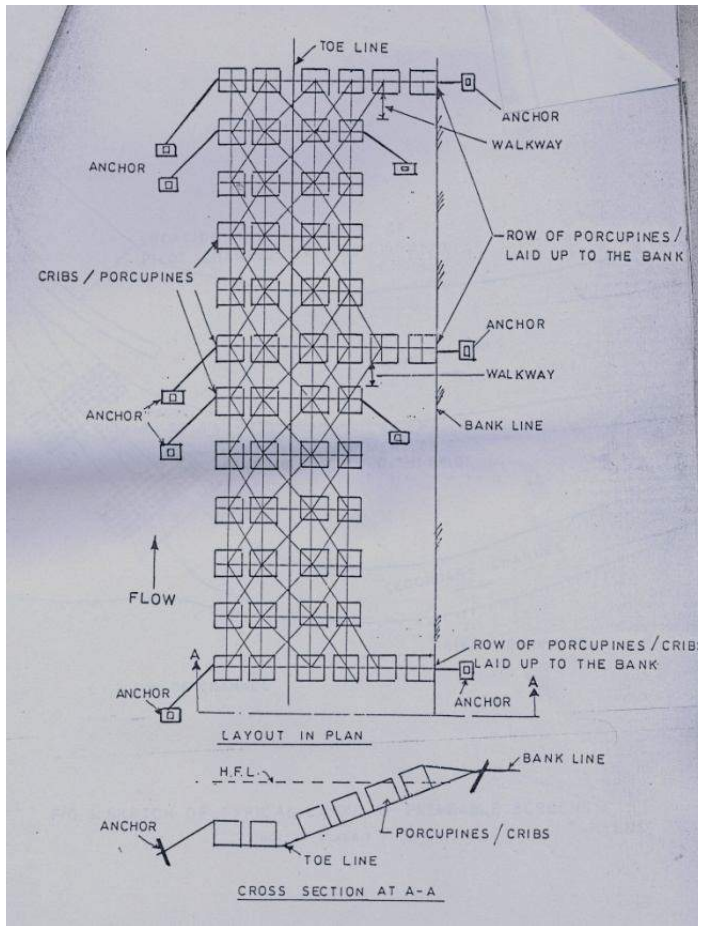


Figure 6-5: Typical RCC porcupine dampener

6.2.4.2 RCC porcupine screens

(a) The RCC porcupine screens are used to choking the secondary channels.

(b) Each porcupine screen is made up of 5 to 9 rows of porcupines (Higher rows for higher flow). The spacing of rows is kept at same as spacing of each porcupine in each row (3 m or 2 m c/c depending upon size of the member).

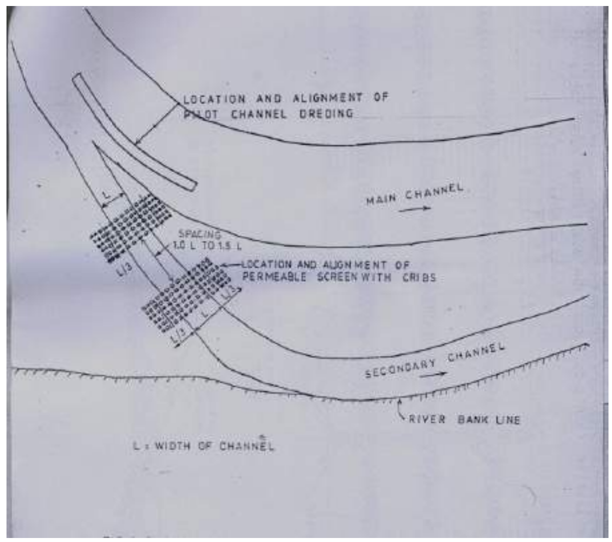
(c) At least two screens are provided to choke the secondary channel. A single screen is generally not found effective.

(d) One screen is normally provided at the entrance of the secondary channel. The second screen is provided at a distance of 1 to 1.5 times width of the secondary channel.

(e) The screens are constructed covering a part or the whole width of secondary channel. If the screen covers the whole width, the screens are extended on both banks for a length 1/3rd of the channel width.

(f) Depending upon the importance, the possibility of development of bypass channel, a third screen can also be provided further d/s at a suitable location.

(g) If the screens are located near the bank, the extension towards bank should be restricted to the design HFL.



Typical sketch of RCC porcupine screen is as under in Figure 6-6.

Figure 6-6: Typical RCC porcupine screen

6.2.4.3 Protection to the permeable structures

(a) The elements ie RCC porcupine members may be tied with each other by GI wire or Nylon ropes (UV stabilized). The tie ropes may be duly anchored to the bank and at the nose with the help of suitable anchor. Depending upon the length of the screen/spur/dampeners, intermediate anchors may be provided at an interval of 15 m to 20 m along the length of structure on u/s side.

(b) No bed protection is needed for the RCC porcupine structures. Sinking of these structures in riverbed is a welcome feature, which adds up to the stability during floods resulting in better performance.

(c) In order to divert the flow and reduce pressure on the RCC porcupine works, wherever feasible, pilot channels should be provided in additional to the river training works.

6.3 Limitation of RCC

(a) In case of high velocity flows, implementation of only RCC porcupine works is not favored. However, use of RCC porcupine works in between the reach of two solid boulder spurs is more effective.

porcupines	(b) Generally additional quantities of RCC porcupines is kept for placing the RCC porcupines in 2nd year or consequent years at locations where partial silting has taken place after implementation of RCC porcupines in 1st year. In the absence of placing additional porcupine the silted region near the bank may not be made firm.
6.4 References	Guidelines for planning and design of Permeable structures in alluvial rivers.

Section-7: Design of Drainage improvement works

7.1 General

IS code 8835:1978 stipulates that Drains are constructed with the object of relieving excess water from agricultural and other areas and disposing of surplus water not required for normal agricultural operations. The proper disposal of surplus rain water is also essential to avoid its percolation down to the water level which may otherwise lead to rise in the water table thereby aggravating or creating the problem of water logging.

The drains may be natural or artificial. As per accepted principles, these are generally aligned along the valley lines between ridges. However, in some cases in order to reduce the length of the drain or to have proper outfall conditions, the drains are taken across valleys. These are known as forced or diversion reaches.

Task Force on Flood Management /Anti Erosion Measures (2004) also stipulated that Water logging due to poor drainage system particularly in flat lands one of the reasons of flood problem in the area resulting in damages to crops and adverse impact and other activities. Providing adequate drainage system where natural drainage system is insufficient, is the basic requirement to lessen the distress caused by floods in the area. A cross drainage work is shown in Figure 7-1.

Figure 7-1: A cross drainage work

7.2 Planning of drainage improvement

Various aspects related to planning of drainage/Channel Improvement projects including data requirement, degree of protection, classification of drains, alignment of drain, capacity of drains etc.

works

7.2.1 Requirement of data

7.2.1.1 Topographical data

7.2.1.2 Hydrological and meteorological data

7.2.1.3 History of past drainage congestion

7.2.2 Degree of protection

are described in paras below:

For the planning of the Drainage/Channel Improvement following data is required.

Index plan showing area affected, bank slope, type of soil, cropping pattern, catchment area, plan and section of earlier executed works.

Discharge, gauge, flow depth, silt content, velocity, cross sections and long section of river, rainfall data etc.

Area of submergence, extent of submergence, duration of submergence etc.

IS Code 8835:1978 stipulates that drains may be designed for 3 day rainfall of 5 year return period. However, in specific cases requiring higher degree of protection,

Return period of 10 or 15 year may also be adopted. Adoption of higher return period rainfall should be justified in term of economics. Cross drainage works should be designed for 3 day rainfall of 50 year return period. Construction of cross drain under the embankment is shown in Figure 7-2.

Figure 7-2: cross drain under the embankment

7.3 Classification of drains

IS Code 8835:1978 stipulates that the drains are broadly classified into the following categories according to the purpose for which these are constructed:

(a) *Outfall drains*- These are the main drains out falling into a nallah or a river from a particular catchment.

(b) *Link Drains*- These are branch drains draining sub-catchment into the outfall drain. These are aligned along subsidiary valley lines.

(c) *Field drains*- These are small drains draining individual or a group of fields into the link drains.

(d) *Ditch drains*- These are constructed to drain the water by connecting borrow pits along roads, railway lines, etc.

(e) *Cunnette*- This is a small drain constructed in the bed of main drain at level lower than the normal bed levels of the main drain for carrying non-monsoon/seepage discharge without allowing it to spread across the entire section of the main drain.

(f) *Seepage drains*- These are constructed along the canals to collect the seepage water from the canal embankments and to drain it either directly into a natural outfall or into a carrier drain.

An embankment with sluice is shown in Figure 7-3.

Figure 7-3: Embankment with sluice

7.4 Alignment of drains

IS code 8535:1978 envisages following guidelines for the alignment of the drainage channel:

The drains should generally follow the drainage line ie. the lowest valley line. As far as possible the alignment of the main or outfall drain should be in the centre of the area to be drained. If the alignment crosses any depressions, ponds or marshes, the drain should not pass through these, as apart from the difficulties in excavation, it affects the hydraulic performance of the drain. In such cases, it is preferable to take the drain away from the depression or pond, and suitably connect it to the drain if it is required to drain the pond or depression.

In selecting alignments, care should be taken to see that as far as possible these do not pass through village habitation. In the forced reaches, care should be taken to see that the embankments of the drains are not of an excessive height in order to minimize the danger of flooding in the event of breaches in the embankments.

As far as possible, the alignment of the drain should be such that the full supply level is below the natural surface level. A sluice with gates is shown in Figure 7-4:

Figure 7-4: A gated sluice

7.5 Capacity /design discharge of drains

IS code 8535:1978 envisages following guidelines for the design discharge of the drainage channel:

Normally the drain is provided to accommodate the design discharge where drains follow natural valley lines. In such cases, no embankments should be provided along the drain so as to allow free flow of water from the surroundings areas. Wherever embankments are necessary for accommodating a portion of the design discharge or where disposal of excavated soil will be very costly, large gaps should be provided in the embankments on either side so as to allow unrestricted inflows, and in case of discharges higher than the channel capacity, the water should spill over the area and return to the channel freely when the discharge in it recedes. In the forced or diversion reaches, embankments on both sides are, however, provided as the design discharge cannot be accommodated within the cut section of the drain. However, even in such cases attempts should be made by selecting a proper alignment to keep the height of the embankments to the minimum. In such cases, inlets of adequate size should be provided in the embankments to admit the water from surrounding areas. A typical drain is shown in Figure 7-5.

Figure 7-5: A typical drain

Intensity of Rainfall- Analysis of the storm rainfall throughout the country indicates that generally the duration of the storm is about 3 days. Therefore, for design of the drains, a storm rainfall of 3 day duration should be taken.

7.5.1 Design frequency of rainfall

In fixing the design capacity of the drain the following factors have to be taken into account:

a) *Economics-* Drains of a bigger size or catering for a rainfall of infrequent occurrence prove to be costly compared to the benefits. Drains are never designed to cater for the worst conditions. In other words, in any drainage project, occurrence of damage at periodical intervals is to be accepted.

b) *Performance-* The experience indicates that drains of a bigger size tend to deteriorate fast, as these are not required to carry the design discharge frequently. Consequently in carrying smaller discharge, drains tend to get silted soon. On the other hand, Drains of a smaller size remain in a better condition and can occasionally carry higher discharges with marginal scour of bed and sides and encroachment on free board.

c) *Land requirement*- On account of small land holdings, bigger drains involve larger land acquisition resulting in a permanent loss of the cultivated land.

d) *Design frequency*- Generally the drains should be designed for three day rainfall of 5 year frequency. Studies carried out indicate that 5 year frequency gives optimum benefit cost ratio. However, in specific cases requiring a higher degree of protection, the frequency of 10 or 15 year can also be adopted. Adoption of such higher frequencies will need to be justified in terms of the economics.

7.5.2 Period of disposal

The period of disposal of the excess rainfall is entirely dependent on the tolerance of individual crops. Crops Like paddy can generally stand submersion for a period of 7 to 10 days without suffering any significant damage. Therefore, in paddy growing areas, the drainage should aim at disposing of the rain water in a period varying from 7 to 10 days. Based on experience the following periods of disposal are recommended:

#	*Crops*	*Period of Disposal*
(i)	Paddy	7 to 10 days
(ii)	Maize, bajra and other similar crops	3 days
(iii)	Sugarcane and bananas	7days
(iv)	Cotton	3 days
(v)	Vegetables	1 day (in case of vegetables, 24 hour rainfall will have to be drained out in 24 hours)

7.5.3 Run-off

Run-off coefficients depends on the type of soil, crops, general topographical conditions like land slopes, etc. In plain areas, the run-off percentage is generally of the order of 15 to 20. In semi- hilly areas the percentage may be higher. Until precise data becomes available, the following run- off coefficients for different soils are recommended for plain areas:

#	*Type of catchment*	*Run-off Coefficient*
(i)	Loam, lightly cultivated or covered	0.40
(ii)	Loam, largely cultivated and suburbs with gardens, lawns, macadamized roads	0.30
(iii)	Sandy soils, light growth	0.20
(iv)	Parks, lawns, meadows, gardens, cultivated area	0.05-0.20
(v)	Plateaus lightly covered	0.70

(vi)	Clayey soils stiff and bare and clayey soils lightly covered	0.55

7.5.4 Run-off for composite crops

In large areas, there are often different types of crops grown. In such cases, the field and link drains can be designed on the basis of the crops grown in a particular area. For the outfall drain, either a composite discharge can be worked out or the total discharge can be worked out by taking into account the discharges from individual link drains. As the area grows larger, the chances of synchronization of discharge from the entire area become less. As such, working out a composite discharge may also serve the purpose. However, individual cases will have to be studied on their own merit. A typical gated sluice for high embankment is shown in Figure 7-6.

Figure 7-6: A typical gated sluice drain

7.6 Capacity/ design discharge for cross drainage works

IS code 8535:1978 stimulates that Cross drainage works are always designed for a higher discharge than the cut sections of the drains. This is mainly on account of the fact that the damage caused to the structures in the event of flows resulting from rainfall higher than the designed rainfall, can be much more than to the drain. Besides, any remodeling of the structures at a later date for higher discharges will not only be costly but time consuming, apart from involving dislocations to facilities like roads, railways, irrigation canals, etc. The drains can, however, be remodeled without much dislocation. The present practices vary considerably.

All the cross drainage structures should, therefore, be designed for a 3day rainfall of 50 year frequency, time of disposal remaining the same depending on the type of crop. In fixing the waterways, care should be taken to see that afflux is within the permissible limits.

7.7 Design discharge for cross drainage works

The drain should be designed as per Lacey's regime theory so that no silting/scouring is occurred in the drain section. Design procedure for the drainage channel may be done as per design of irrigation channel by Lacey's theory. The design procedure is as under:

> Velocity of the flow (V in m/sec) = $(Qf^2/140)^{1/6}$
> Where Q = design discharge in cumec and, f is the silt factor, which can be worked out using the formula f = 1.76 (d) $^{1/2,}$ where d is the average bed material size in mm
> Hydraulic mean depth (R in m) = 2.5* (V^2/f)
> Area of channel section (A in m2) = Q/V
> Wetted perimeter (P in m) = 4.75 (Q) $^{1/2}$
> and Bed slope (S) = (f $^{5/3}$) / (3340*$Q^{1/6}$)

IS code 8535:1978 envisages following guidelines for the design of the drainage channel.

7.7.1 Velocity

The drain section shall be adequate to carry the designed discharge and the velocity shall be non-silting, non-scouring to be determined by Manning's formula.

7.7.2 Discharge capacity of the drain

In order to obtain the discharge capacity of a drain it is necessary to know the mean velocity of flow as obtained above which when multiplied by the area of the cross section of the drain in square meters will give the discharge in m^3/s.

7.7.3 Side slopes

In selecting the side slopes for the drain, it will be necessary to consider the kind of material through which the drain is to be excavated. Generally side slopes of 1.5H : 1V are provided.

7.7.4 Cross sections of the drain

Although deeper sections of the drain may be desirable, the width to depth ratio should be so selected that the section is both hydraulically efficient as well as economical in excavation. In the case of drains with embankments, the berm width equal to the depth of the drain, subject to a minimum of 1 m should be provided between the toe of the embankment and the section of the drain. The top of the embankments should be 1 m higher than the design full supply level. Wherever, there is likelihood of backing up effect on account of floods in a river into which the drain outfalls, the top of the embankments should be so designed that the flood levels on account of back water conditions are accommodated within the section over which the minimum freeboard is to be provided.

7.7.5 Fixation of full supply level (FSL) at outfall

Whenever the drain is out falling into a river, the FSL should be slightly higher than the dominant flood level. The dominant flood level is the stage of river/outfall which is (a) attained and not exceeded for more than 3 days at a time; and also (b) attained and not exceeded 75% of time over a period of preferably not less than 10 years. In cases where the topography permits, the FSL can be above the highest flood level. However, if such a level results in flatter slopes or in FSL becoming higher than the natural ground level, FSL at outfall should be kept slightly above the dominant flood level. In such cases, there will be backing up in the drain when the river rises above the dominant flood level. Such occurrences being infrequent and of short duration can be tolerated. Care shall, however, be taken in determining the dominant flood discharge and the level.

7.7.6 Hydraulic slope

The FSL of the drain as far as possible should be at or below the ground level. Where it cannot be ensured, the FSL should in no case be more than 0.3 m above the average ground level at the starting point of the drain. The hydraulic should then be determined adopting the stipulation and the criteria laid down for fixation of FSL at outfall. The hydraulic slope should normally be such as to provide permissible velocities as indicated in VII.7.1 above.

7.7.7 Tidal lockage

In the case of drains out falling into rivers subjected to tidal influence, the reaches of the drains which will be subjected to tidal lockage should be determined. In these reaches capacity of the drains should be increased to provide for duration of the tidal lockage gradually diminishing from the outfall towards the upstream. For this purpose, it will be necessary to plot the dominant tidal curves. The FSL of the drains in such cases should normally be fixed at mean tide levels. This will also be known as cut off level. This will be level at which the drain will again start discharging during the ebb tide. This level will always be higher than the cut off level. In major outfall drains, an outfall regulator should invariably be constructed to prevent tides entering the channel, which will result in silting of the drains.

7.7.8 Falls

Normally no falls are to be provided in drains except in rare cases where there is a sudden appreciable drop in the natural surface level or where the FSL s likely to be more than natural surface level without provision of falls.

7.8 Longitudinal section

IS code 8535:1978 envisages following guidelines for deciding the longitudinal profile of drainage channel.

7.8.1 Collection of data

The following data should be collected while carrying out surveys along different alternatives alignments of drains:

(a) Cross sections at every 150 meters to the full land width.
(b) Natural ground, design bed and full supply levels at every 150 meters.
(c) Locations of inlets of link/field drains with related hydraulic data.
(d) Full data of all crossings like roads, railways, irrigation canals, etc.
(e) Representative soil samples to determine the probable stable side slopes.
(f) Ground water levels at a distance of about 2 kilometers.
(g) Boundaries and slopes of the areas needing drainage.
(h) Existing drains.
(i) Location and elevation of all depressions, drains, mounds and ridges.
(j) Location and elevation of possible inlets (outfalls).
(k) Area that will drain into each part of the system.
(l) Flood data of Outfall River and study of backwater effect of flood.

7.8.2 Preparation of longitudinal section

(a) Fix outfall level considering the dominant flood levels in the river/drain and the likely backing up.
(b) Hydraulic slope to be determined on the basis of the ground levels, permissible submersion and the outfall levels determined in (a).
(c) Plot the natural ground levels, design bed levels, full sully levels and the backwater profiles, if any.
(d) Divide the drain in convenient reaches between inlets sites o the junction of tributary/link drains. The capacity of the drains in each of these reaches should be uniform. The capacity will change with the addition of discharge from tributary/link drains

7.9 Channel improvement by dredging

The method of improving the channel by improving the hydraulic conditions of the river channels by desilting, dredging, lining etc., to enable the river to carry its discharges at lower levels or within its banks has been often advocated but adopted on a very limited extent because of its high cost and other problems especially because the success of this method of river improvement for lowering flood levels depends on outfall conditions which can not be changed appreciably.

Dredging operations of the Brahmaputra, which were undertaken in the early seventies on an experimental basis, were discontinued because of their prohibitive cost and limited benefits. Dredging in selected locations may perhaps be considered as a component of a package of measures for channel improvement to check the river bank erosion subject to techno-economic justification. It may be economically justifiable as a method for channel improvement where navigation is involved. Dredging is sometimes advocated for clearing river mouth or narrow constrictions.

The Task Force on Flood Management /Anti Erosion Measures (2004) also recommended that "*desilting operations may be carried out for improvement of carrying capacity of drainage channels downstream of sluices at their outfalls into rivers and in upstream selectively, if absolutely essential. Desilting at mouth of the tributaries out falling into main rivers, at the mouth of rivers out falling into sea and at other critical locations may also be taken up selectively after detailed studies".*

7.10 Methodology for determining the dominant flood level: an illustration

As per example given in IS Code 8835:1978 the method for determining the dominant flood level has been explained in table below:

The record of gauge hydrographs of a river, as available from 1962 to 1971. The levels which were attained and not exceeded for more than 3 days at a time are as under:

Year	*Levels attained and not exceeded for more than 3 days at a time*						
1962	138.30	137.20	138.00	135.75	125.00		
1963	137.85	136.85	135.00	132.00	129.60		
1964	137.30	132.10	123.60	121.00			
1965	137.30	132.10	123.60	121.00			
1966	136.45	135.30	128.75				
1967	135.60	133.10	133.00	130.45	128.62	120.15	
1968	137.00	135.62	135.45	132.45	130.46	128.62	120.15
1969	137.60	136.60					
1970	139.30	138.31	138.60	133.30	122.75		
1971	142.45	142.30	139.28	128.55	129.00	114.00	

To determine the level which is attained and not exceeded for 75% of the time, the above levels are arranged in ascending order:

Event	*Level*	*Event*	*Level*	*Event*	*Level*	*Event*	*Level*
1	114.00	13	129.60	25	135.00	37	137.30
2	120.15	14	130.45	26	135.30	38	137.60

3	121.00	15	130.46	27	135.45	39	137.85
4	122.75	16	132.00	28	135.60	40	138.00
5	123.60	17	132.10	29	135.62	41	138.10
6	124.45	18	132.45	30	135.75	42	138.30
7	125.00	19	132.47	31	136.45	43	138.31
8	128.55	20	132.60	32	136.50	44	138.60
9	128.60	21	133.00	33	136.60	45	139.28
10	128.62	22	133.10	34	136.85	46	139.28
11	128.75	23	133.30	35	137.00	47	142.30
12	129.00	24	133.75	36	137.20	48	142.45

The level, which is attained and not exceeded 75% of time is at event no. 48x75/100= 36

Hence dominant flood level (at event no. 36) = 137.20 m

7.11 Design of regime drainage channel using the Lacey's theory: an illustration

Assuming design discharge Q= 50 cumec and silt factor f= 1.1.

Velocity $V = (Qf^2/140)^{1/6}$
$= (50*1.1^2/140)^{1/6}$
$V = (50*1.21/140)^{1/6} = 0.8695$ m/sec
Area of cross section (A) = Q/V= 50/0.8695= 57.50 m^2
Hydraulic mean depth (R) = $2.5* (V^2/f)$
$= 2.5* (0.869^2/1.1)$
= 1.72 m
Wetted perimeter (P in m) = $4.75 (Q)^{1/2}$
$= 4.75 (50)^{1/2}$
= 33.59 m

For a trapezoidal channel with side slope of 0.5H:1V
$P = b+2*d (0.5^2+1^2)^{1/2}$
where b = width of channel in m and d is depth of channel in m
P = b+2.24d
33.59 = b+2.24d or b= 33.59-2.24d
A = (b+0.5d) *d
$57.50 = bd+0.5d^2$
$(33.59-2.24d) *d+0.5d^2 = 57.50$
$33.59d-2.24d^2+0.5d^2 = 57.50$
$1.74d^2-33.59d+57.50 = 0$
$d^2-19.31d+33.05 = 0$
$d = (19.31- (19.31^2-132.2)^{0.5}) /2$ (neglecting + sign for realistic values of d)
= (19.31-15.51) /2
= 1.90 m

b = 33.59-2.24*1.9 = 29.33 m
channel slope (S) = $(f^{5/3}) / (3340*Q^{1/6})$
$= (1.1^{5/3}) / (3340*50^{1/6}$
= 0.00018 or 1 in 5459.
Hence design of channel is as under:

Base width (b) = 29.33 m
Depth (d) = 1.90 m
Channel slope (S) = 1 in 5469
Velocity (V) = 0.8695 m /sec

7.12 References

1. IS code 8535:1978: Guidelines of Planning and design of surface drains.
2. Report of Task Force on Flood Management /Anti Erosion Measures (2004).

Section-8: Implementation and Construction Methodology

8.1 General

The construction planning for works envisaged in any flood management/river training works is a vital component for the timely completion of the works avoiding time and cost overrun. Time is of high essence of flood management works as the same has to be completed in available non-monsoon season. Construction Planning becomes part of the overall activity starting from off-setting of monsoon which include (i) vulnerability/damage assessment; (ii) Type of measures to be taken, (iii) Design of structures, construction planning and preparation of DPR, (iv) Administrative approval of DPR and (v) Implementation of the works while keeping sufficient time for each activity, time to accommodate unforeseen issues should also be kept in mind.

Implementation of a flood management/river training works include tenders for various works, site survey like latest river configuration, site clearance etc, mobilization of resources like men, material at the site in pre-organized manner for various river training and flood control works.

Construction methodology includes proper construction of the works as per the approved design and drawings. This also includes the in-time procurement, mobilization and proper installation of the construction materials being used for the works. The installation procedure for the innovative materials like geo-textile bags, geo-mattress, geo-textile tubes etc should be performed in a systematic controlled and well planed manner so that it would give optimum benefits due to use of these materials.

8.2 Construction planning

It is understood that construction planning is the key for in-time completion of the flood management and river training works. It is seen from the past experience that most of the projects are delayed in completion due to lack of proper construction planning.

For a proper completion of a project, the storage and installation of new innovative material for construction of embankments, revetment, spurs etc. for project specific problem need to be executed under well trained guidance and accuracy. The planning for the same needs to be done considering all the situations like working season, monsoon season, land acquisition, site survey and clearance, procurement of materials etc. The Implementation of project may involve following steps:

8.2.1 Tendering

Model tender documents for procurement of materials including Geo-textile bags, Geo-textile tubes, mattress, wire-mesh and for various civil works including earth work, boulder work, launching of RCC porcupines should be prepared and used immediately after administrative approval of the project.

8.2.2 Procurement of construction material

Construction materials required frequently in large quantities including boulders, sand, Geo-textile bags, Geo-textile tubes, mattress, wire mesh should be procured well in advance preferably during monsoon season to save time. Any additional quantities as per approved DPR may be procured concurrent to execution of works.

8.2.3 Storage of construction material at site

There should be proper space/shed for the storage of construction material. The storage space/shed should be such, there is no risk of wear-n-tear and theft of the construction material till the works are over.

8.2.4 Testing of the material

There should be arrangement of testing of the construction material before the start of the work. Provision for standard testing along with procedure of testing should be made a pet of the tender document. All the construction material should posses qualifying standards before construction.

8.3 Construction methodology

The construction methodology and equipment planning for various works is based on the site conditions prevailing in the project area. Construction activities are planned in such a way that the project will be completed in the shortest possible time period. The activities are divided into pre construction and construction/ main (during construction) activities. The pre construction activities like land acquisition, infrastructure works and procurement, transportation and testing of materials are completed before the start of construction works. All the main activities for the project will be executed in phases and some works will be executed simultaneously.

The detailed design may be done in parallel with the pre-construction works. During tendering stage, detailed design work may be started and construction drawings may be available by the time contracts are awarded.

Detailed construction guidelines for the works using the like gabions, geo-textile tubes and bags etc are given below.

8.3.1 Construction methodology for gabion/ crate structures

As per construction practices adopted by the M/s Maccaferi India Pvt Ltd., construction methodology for gabion structures is as under:

Gabions are a cage made from mechanically woven Double Twisted (DT) steel wire mesh. They are uniformly partitioned, of variable size, interconnected with other similar containers and filled with stone at the site of use, to form flexible, permeable, monolithic structures such as spurs, retaining walls etc for anti erosion works. These gabions are filled by the boulders, which are obtained from the nearby quarry. Boulders Rocks should be hard, angular to round, durable and of such quality that they do not lose their integrity on exposure to water or weathering during the life of the structure. The size of the rocks should range between 150 and 300 mm A gabion retaining wall is given in Figure 8-1 as under:

Figure 8-1: Gabion retaining wall

The following sequence shall be followed in the construction of Gabion/crates structures:

i. The natural surface should be excavated to the required level as per the drawings for formation of the base of the structure. The surface of formation should be free from any deleterious material and unwanted foreign objects. Loose pockets if any, should be excavated and filled with suitable granular or backfill material.
ii. The excavated surface should be compacted by using the Vibro-roller of 8 to 10 tonne capacity. The design requirements with respect to bearing capacity should be achieved and verified before proceeding further. Density of compacted formation should be greater than or equal to 95% of the modified Proctor value.
iii. The excavated surface prepared should be levelled without ruts and undulations and Geo-textile filter material laid over this as shown in Figure 8-2 and

Figure 8-3 below:

Figure 8-2: Formation of excavated surface

Figure 8-3: Geo-textile laid out on rolled surface

iv. The folded gabion unit may be opened by placing it on a flat surface or leveled ground and straightening out the various end panels as shown in the Figure 8-4. One or two persons may be deployed for the straightening of DT mesh panel activity depending on the size of gabions.

Figure 8-4: Straightening out of DT mesh panels of folded gabion unit

v. The back, front, end panels may be folded along the lines. The gabion box may be assembled by making the end panels on either side meet with the front and back panels. The resulting arrangement is as shown in the Figure 8-5 below.

Figure 8-5: Straightened out Gabion with diaphragm fixed in position

vi. The gabions may be placed at the leveled surface over the geo-textile layer. All the gabions placed may be connected to each other. The resulting arrangement is as shown in the Figure 8-6 below:

Figure 8-6: Laying and connecting all adjacent gabions

vii. The top and bottom ends of the panels may be stretched along the longitudinal direction as shown in the Figure 8-7. This arrangement will keep the front and back panels in tension during the rock filling operation.

Figure 8-7: Stretching the gabions

viii. Boulders for filling the gabion may be obtained from nearby quarry. The boulders may be selected in such a way that at least one face is flat-shaped. Boulders of smaller size (40 mm to 100 mm) may be used for packing the voids and reducing the porosity. Boulder filled gabion is shown as under in Figure 8-8.

Figure 8-8: Boulders filled in gabions

ix. The boulders may be placed in lifts of 300 mm. The filled layer should never be more than 300 mm higher than any adjoining cell. To avoid such circumstances the filling pattern shown in **Error! Not a valid bookmark self-reference.** may be followed.

x. After the filling of gabion, the same may be tied securely so that boulders filled inside the gabions are kept intact. It is also recommended to slightly overfill the baskets by 25 to 50 mm to allow for settlement of the boulders.

xi. Geo-textile layer for filter purpose may be kept between the gabions and natural soil so that soil particles can't get out of the natural soil due to suction. This will also prevent the structural fill from being washed into the rock voids in the event of a rainfall and to drain off excess water from the structural fill.

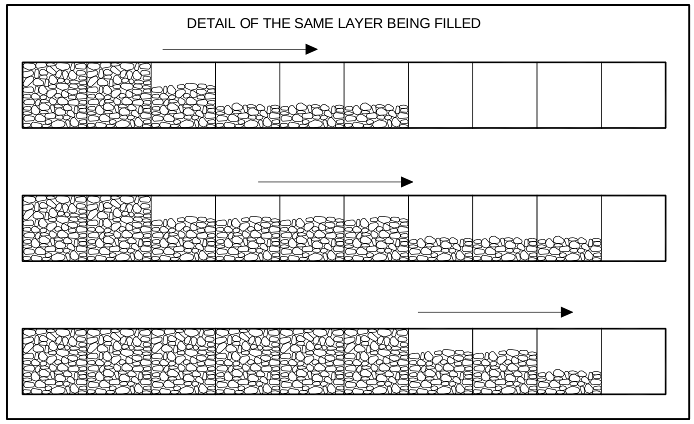


Figure 8-9: Filling pattern of gabions boxes

8.3.2 Construction methodology for anti erosion measures with geo-bags

As per construction practices adopted by the M/S Maccaferi India Pvt Ltd, construction methodology for geo-bag structures is as under.

Geo-textile Bags are made of woven or non woven geo-textile fabrics. It is specially designed for good soil tightness and high seam efficiency. Generally non-woven geo-textiles material is used for filter media purpose and woven geo-textile is used for geo-bags which serve the purpose of anti erosion measures. Geo-textile Bag is a container made of permeable woven geo-textile which can be filled with sand. A typical geo-textile Bag is shown in Figure 8-10.

Figure 8-10: Geo-textile bag

The following sequence shall be followed in the construction of geo-bags structures.

i. The site survey and bathymetric survey for the affected reach, where the geo-bag protection is proposed, may be carried out.

ii. The river bank may be trimmed to achieve the required slope mentioned in the project drawing. Excavation may be mostly carried out by track mounted excavators. Lighter excavators such as JCB may be employed if required.

iii. The excavated earth may be disposed of by tippers/tractors to a suitable distance from the bank sides, so as to reduce the overburden on the existing bank.

iv. The loose earth residue remaining after the excavation by the machineries may be removed manually. The slope formation may be given necessary ramming to remove any undulations and corrected to the desired slope (if required) before the start of installation of the non woven Geo-textile on the trimmed surface. At the bottom and top of the slope, anchor trench or key shall be prepared as per the required design.

v. The geo-textile fabric may be laid across the dressed slope over which sand filled geo-textile bags may be kept.

vi. The geo-textile may be filled at a location preferably higher than the HFL. This may ensure that the filling process of the bag remains unaffected by the rain and subsequent changes in the water levels.

vii. The sand used for filling may be transported to the site by dumpers and bulldozers or any other appropriate mode conducive to the local availability and geographical suitability. The bags may be supplied to the project in the folded form and packed in bundles. The bags may be filled with sand by suitable methods. The sand used for filling shall conform to the technical specifications of the project. Filling of the sand bags is shown in Figure 8-11.

Figure 8-11: Filling of geo-bags

viii. After the sand filling is completed, the geo-textile bag may be weighed to confirm the desired weight as per the technical requirements of the project. After confirmation of the weight, the geo-textile bag shall be stitched at the mouth by means of a Bag closing machine as shown in the Figure 8-12.

Figure 8-12: Stitching of geo-textile bag

ix. For the sake of easy counting and monitoring the geo-textile bags shall be stored in batches of 100 numbers or any suitable multiple as per the available space at site. The filled bags in group are shown in Figure 8-13.

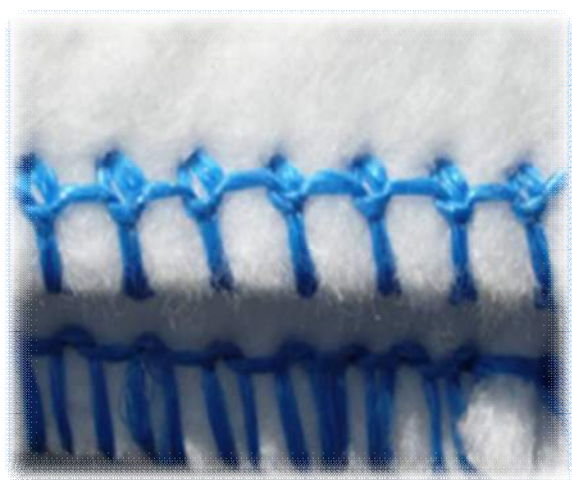


Figure 8-13: Filled Geo-bags

x. The dumping of Geo-bags for launching apron may be carried out using the motor boat/barge. The motor boat/barge may be positioned at the required location with the help of Total Station and anchored in position with the help of anchors. The dumping of the geo-textile bags shall be continued up to the location of the key at the bottom of the slope near LWL.

8.4 Construction practices by Maccaferri India Pvt

Limited.